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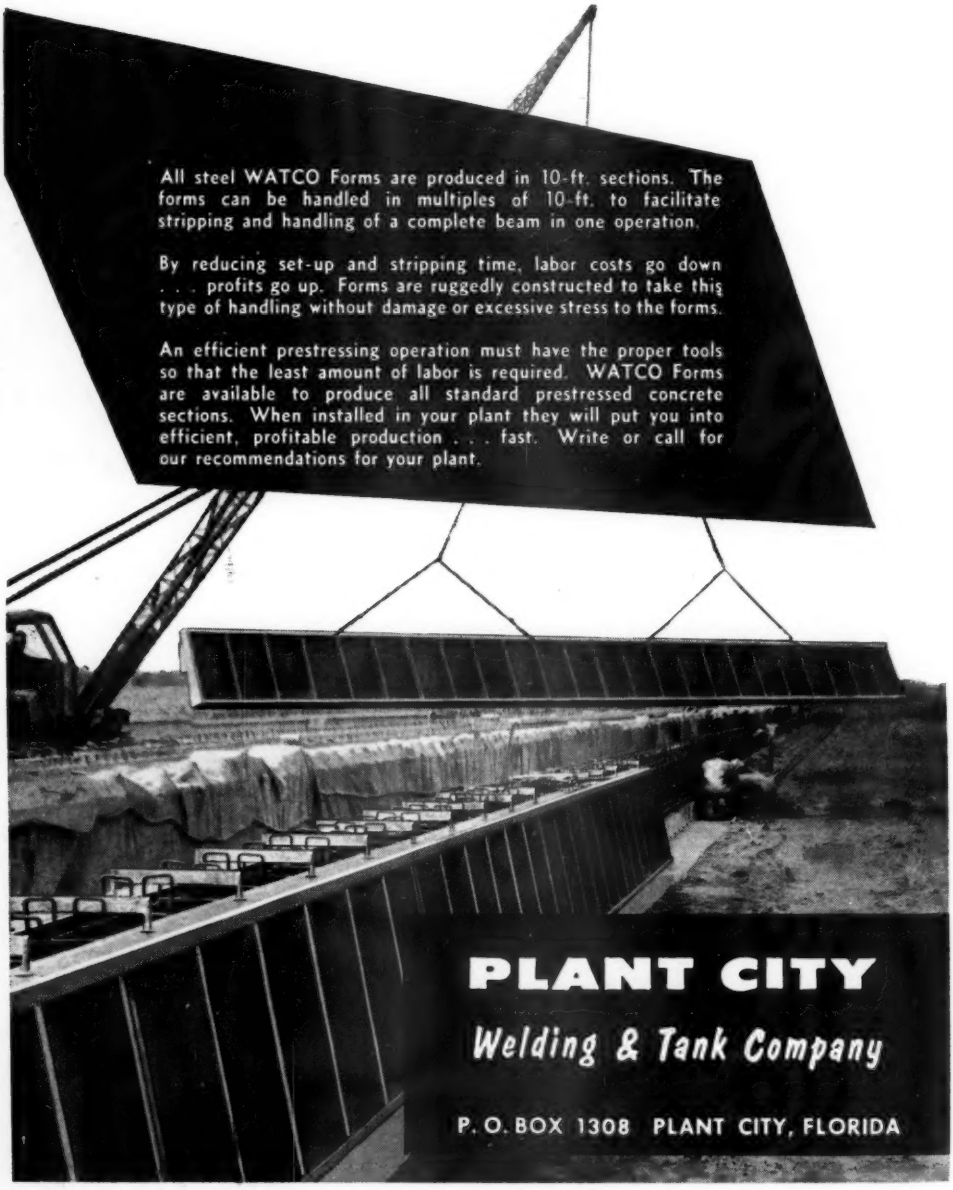
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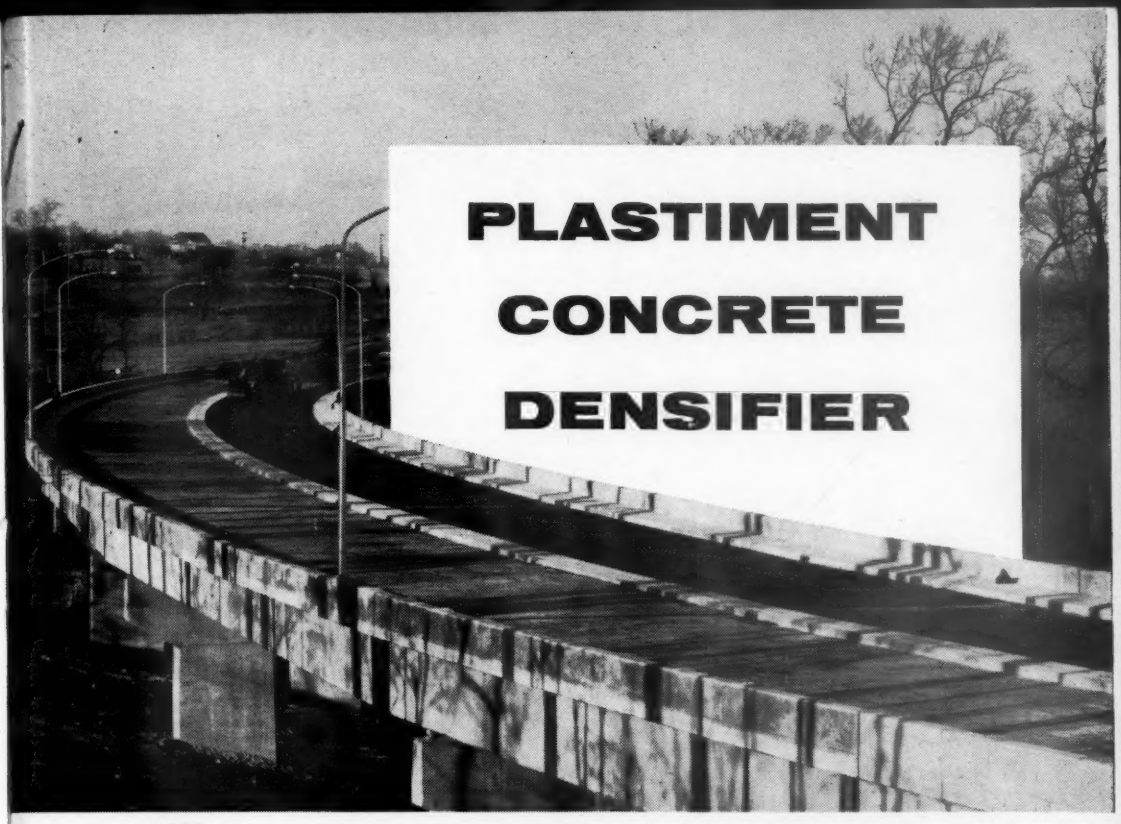

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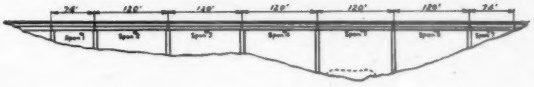

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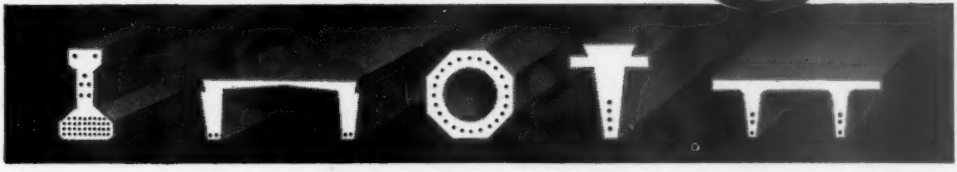

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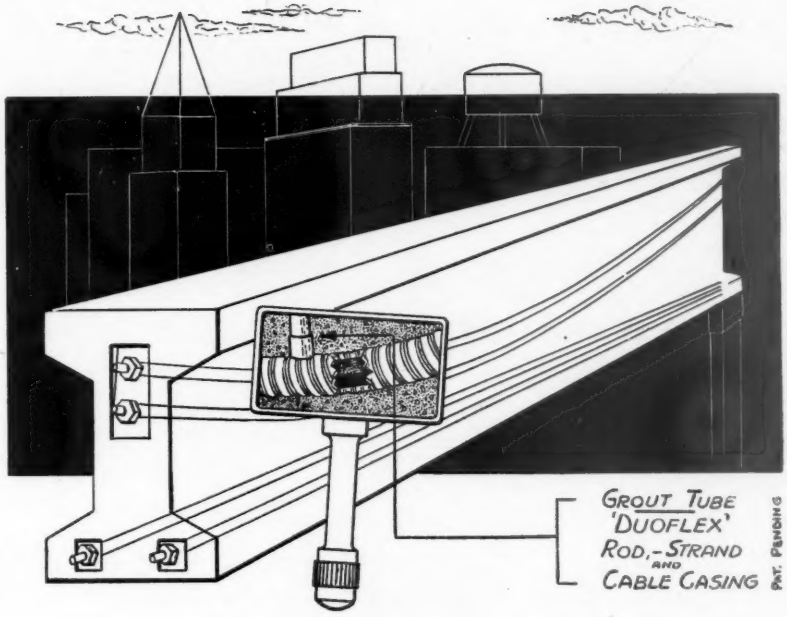

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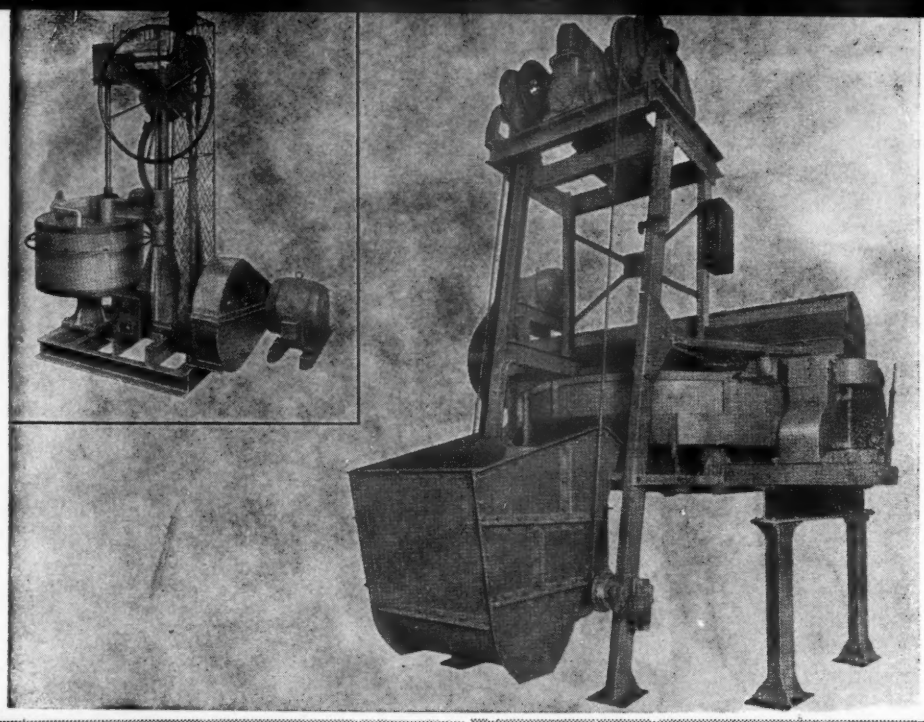

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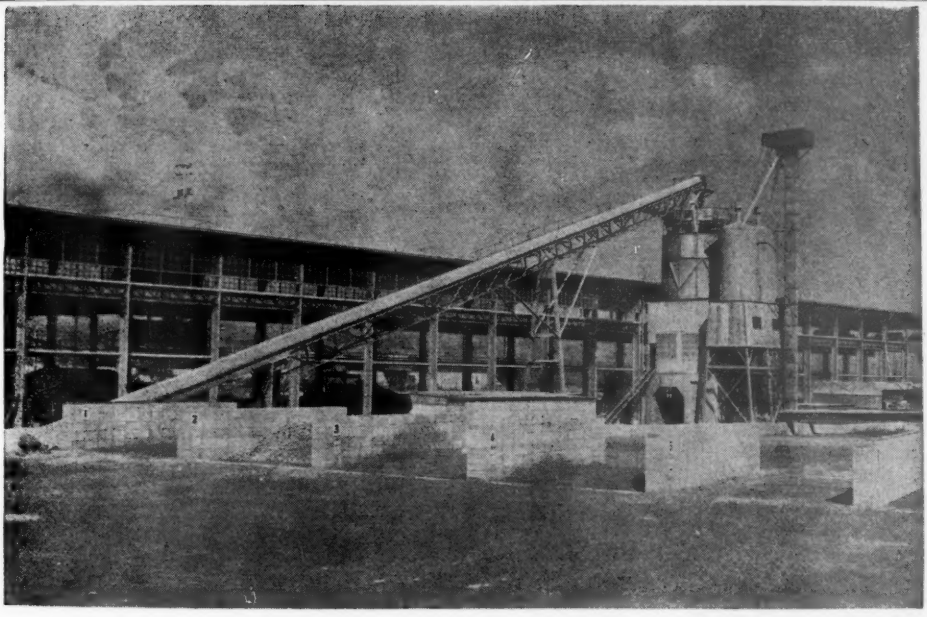

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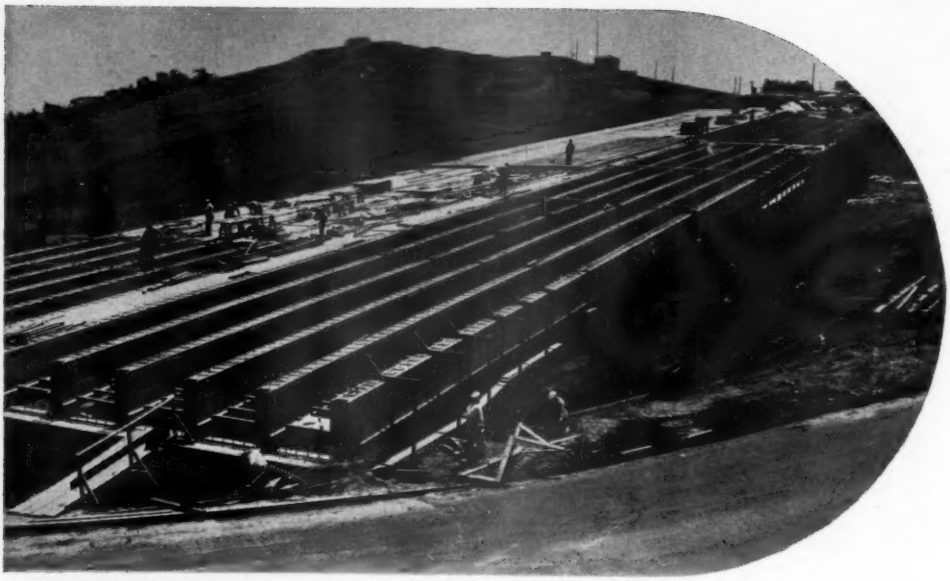

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From the editor. . .

The complete dependence of the industry on quality in the design and manufacture of structural units again was reiterated at the Second National Prestressed Concrete Short Course at Ellinor Village, Daytona Beach, Florida, January 27 to 31. Speaker after speaker emphasized that the future success of prestressed concrete depends on continuous improvement.

The ever growing interest in the industry was indicated by the registration of about 400 people from 40 states, Canada, the British West Indies and Puerto Rico in the course co-sponsored by the Civil Engineering Department of the University of Florida at Gainesville, and The Prestressed Concrete Institute. And the most encouraging part of the meeting was their enthusiastic desire to learn more about the various phases of the industry.

The response to the short course emphasizes again that as long as engineering and research activities keep their present pace and as long as quality products are manufactured by competent producers, prestressed concrete will become ever more important among structural materials.

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MAGIC OR

By Col. Martin P. KORN
Executive Secretary
Prestressed Concrete Institute

Dramatic Demonstration of Prestressing Amazes Spectators At Beam Test

Test described conducted by Prof. A. M. Ozell, College of Engineering, University of Florida, Gainesville, Florida

It happened near Jupiter, Florida, not too many months ago when Juno Prestressors put on a show. A beam was to be tested. Was that the only attraction . . . just a test?

A beam is a beam and there is nothing new in that! But this test was to be different, for many were curious, or skeptical, or both. And so they came — architects, engineers, contractors and builders.

The beam was a 14" prestressed concrete channel section, 2'-6" wide, 63'-0" long, original camber plus 4", design live load 30 lbs. per sq. ft. The beam was loaded with one live load and down went the beam,. the camber reduced to plus 1⅝".

A second live load was added and down went the beam again, now registering a deflection of -1⅜". The beam, loaded with 60 lbs. per sq. ft., had gone down 5⅜" at its center seemingly like rubber, not a crack in sight. Curiosity was ripe.

What if a third live load were added? Would they dare? Any beam of steel or reinforced concrete would hardly be expected to look happy under two live loads. Certainly not reinforced concrete which would surely show cracks. But three live loads — was not that asking too much of prestressed concrete?

The producer did not think so nor was he backing away from a challenge. So the

This article was originally published in the

REALITY

word was passed around that a third live load would be added. And the loading began.

Cement blocks one after another distributed symmetrically, went to work, the beam went down and down, an inch, and another inch; finally more than one foot. The crowd stepped back. What if the beam collapsed? Anything could happen. Everyone seemed primed for the climactic moment — a drama of stress and strain and then . . . !

Silence, suspense, anxiety! Breathing seemingly stopped. The third live load was completed. Nothing was happening. The beam held fast, the audience with it. A pause and then the group moved close; some cracks had appeared, they were hardly more than hairlines.

Deflection now registered -9", a drop of 13" at center of beam! Talk about rubber, here was prestressed concrete, a structural material, load bearing and certainly putting on a show of strength, its muscles flexed under a live load of 90 lbs. per sq. ft., three times its design load! Now what did the spectators think?

Granted the beam was strong. But just because it stood up under three live loads like rubber didn't mean it was good any longer, or was it?

One live load was removed and the beam recovered from a -9" deflection to 2¾".

A second live load was removed and the beam moved up to show a camber of plus 1⅜". What now?

The third and last live load was removed and the beam seemed to bounce up in victory — its camber restored to plus 3¾". Surprise and thrills reflected in the expressions of the spectators. Not a crack was visible. All had disappeared. Recovery was almost complete. A permanent drop of a mere ⅜" after that herculean display of strength and resiliency. Was it reality or magic in that prestressed concrete?

Reality, as attested by numerous similar tests of varying types of prestressed concrete products around the country. A reinforced concrete beam could not have made it if only 14" deep. Not even a steel beam. Here then was something for the architects, engineers, contractors and builders to ponder.

Prestressed concrete has strength, but it has something more. It had resiliency greater than reinforced concrete or even structural steel, a natural for dynamic loadings. It spells economy in costs, no maintenance, savings in headroom by virtue of its shallower depths. It is available for quick delivery and erection. It offers versatility. Yes, the magic of prestressed concrete is now a reality.

THE INFLUENCE OF CEMENT on creep of concrete and mortar

BY A. M. NEVILLE,
M.C.(POL.), M.SC.(ENG.),
LECTURER IN ENGINEERING,
UNIVERSITY OF MANCHESTER,
MANCHESTER, ENGLAND

INTRODUCTION

Creep is an inherent property of concrete, and no concrete structure subjected to a sustained load is free from the effects of creep. In some reinforced concrete structures creep may be of little importance; in others, it may lead to stress re-distribution, often, but not always, of a beneficial nature. In prestressed concrete, however, creep is of singular importance: it is, in fact, responsible for the failure of all early attempts at prestressing, in which mild steel with a prestress of the order of 30,000 p.s.i. was used. As is well known, it was only when steel with a very high elastic limit, and therefore large elongation at prestress, was used that the losses in prestress due to creep and shrinkage of concrete were reduced to only a fraction of the initial prestress.

The design of prestressed concrete structures requires an accurate estimate of the creep which will take place. The influence on creep of mix proportions and storage

conditions has been investigated in the past but the mechanism of creep of concrete has not been determined with any degree of certainty(1). It is known, however, that it is the cement paste that is primarily responsible for creep. Creep is, therefore, likely to be related to the structure and chemical composition of the hydrated cement, which, in turn, is a function of the chemical composition of the clinker. Correlation has been obtained for some other properties of hydrated cement, and equations in terms of parameters of cement components have been written for strength,(2) heat of hydration,(3) specific surface of the cement paste,(4) and shrinkage(5). It would be reasonable to expect a similar equation for creep. This, it is believed, could be obtained for laboratory-made cements but in the case of commercial cements there are factors additional to the potential chemical composition, notably the rate of cooling of clinker and the resultant glass content. These factors are known also to affect the strength and thus vitiate a simple prediction from the composition of clinker in the case of strength, too.

In this paper, the creep data obtained by various investigators for cement of different types are reviewed. This is followed by a discussion of the results obtained by the author who is continuing a systematic investigation of creep of various cements.

INFLUENCE OF TYPE OF CEMENT ON CREEP IN COMPRESSION

For concrete loaded in air, creep appears to be inversely proportional to the rapidity of hardening of the cement used. Thus creep at the same age will be in an increasing order for concretes made with the following cements: Aluminous, Rapid Hardening (Type III), Standard Portland (Type I), Portland Blast Furnace, Low Heat (Type IV) and Portland Pozzolana. The order of the last three is not definitely established as test conditions were not directly comparable.(6,7) It is known that at the same age these cements have achieved varying proportions of their final strengths when mature: it is logical that the more hardened the paste the more rigid it is and the less susceptible to creep. A better and more practical picture would, therefore, be obtained by comparing creep at ages corresponding to the same relative maturity, e.g. 1 day for aluminous cement and 28 days for Standard Portland (Type I) cement.

The behaviour of Portland Blast Furnace cement concrete was compared fully with that of standard Portland cement concrete.(6) The creep-time curves are of the same charactertistic shape but the former creeps vary much more than the latter. A 1 : 2 : 4 mix concrete with 0.65 water-cement ratio under a stress of 400 psi for 110 days showed a creep of 104×10^{-5} when made with Portland Blast Furnace Cement, and only 63×10^{-5} when made with Standard Portland Cement. The strengths of the two concretes were 2555 and 2905 psi respectively. It is interesting to observe that shrinkage was practically the same for the two concretes: it seems that high shrinkage and high creep are not necessarily concomitant.

A comparison of Low Heat cement (Type IV) concrete with standard Portland cement (Type I) concrete(8) showed that the creep in the former case is some 80 per cent in excess of the creep of Normal Portland cement concrete. This value was found for the same age of the two concretes when both were air-stored and also when they were water-stored.

While standard Portland cement (Type I) concrete creeps in compression very much less in water than in air,(10) the difference in the case of Rapid Hardening cement (Type III) is very much smaller.(11) As a result, the creep of standard Portland cement concrete is greater than that of Rapid Hardening Portland cement concrete when loaded in air, while the reverse is the case when loaded in water.

Very little is known about the creep properties of aluminous cement concrete. The shape of its creep-time curve is different from those of all the Portland cements.(12) All these start at parabolas, then their slope falls off steadily and they approach asymptotically a limiting value. On the other hand, for aluminous cement concrete, the parabolic part is followed by a straight line, i.e. the creep continues at a constant rate during a period of some five years and then the rate decreases steadily, the final creep probably approaching a limiting value.

Some comparative figures for specific creep (per lb. per sq. inch) at one year of

air storage of 1 : 2 : 4 concretes loaded at 28 days are given by Glanville:[10]

Aluminous cement	0.25×10^{-6}
Rapid Hardening cement (Type III)	0.39×10^{-6}
Normal Portland cement (Type I)	1.02×10^{-6}

The behaviour of aluminous cement concrete loaded in water is even more unusual: its creep in water is higher than in air, while the reverse is the case with standard and Rapid Hardening Portland cement concretes. There seems to be a strong case for further and fuller tests on creep of aluminous cement concrete.

INFLUENCE OF TYPE OF CEMENT ON CREEP IN TENSION

Creep in tension is difficult to measure but it of great interest in the case of mass concrete subjected to the usual hydration temperature cycle. It has been found that in Portland-Pozzolana and Low Heat cement concretes,[7] unlike Normal Portland cement, the temperature of the mass drops well below the initial temperature before failure occurs. This indicates that, apart from a lower rate of heat generation, these cements are capable of greater tensile creep.

Standard Portland (Type I) and Rapid Hardening Portland cement (Type III) concretes show practically equal shrinkage but standard Portland cement affords greater relief of stress when restrained. Stress due to shrinkage reaches a maximum value a few months after casting and then decreases, the decrease being more marked in the case of standard Portland cement concrete.[9] This is similar to the effects of different rates of shrinkage. The total shrinkage of a concrete specimen is the same regardless of whether it has been allowed to dry out immediately after casting or after a period of curing. Since the rate of creep is lower the greater the age when shrinkage becomes operative, it follows that the relief of stress due to creep will be less for the moist cured concrete, and consequently, shrinkage stresses in members subjected to prolonged curing and subsequently allowed to dry, may be greater than in similar members allowed to dry out at an earlier age. At the same time, the greater the age the greater the tensile strength of concrete and the greater its extensibility.

INFLUENCE OF FINENESS OF CEMENT ON CREEP

It is possible that creep may be affected not only by the chemical composition of cement but also by its fineness. Unfortunately, on the basis of the data available this effect seems to be complicated and non-uniform for various cements.

In the case of standard Portland cement (Type I) concrete, creep was found to be greater the finer the cement. Low Heat cement (Type IV) concrete was observed to creep more than concrete made with standard Portland cement of the same fineness, but when coarsely ground (1300 sq. cm. per gram) it exhibited a much greater creep and also a greater rate of creep at all ages.[8] This is rather difficult to explain in terms of fineness alone, but it is possible that the gypsum content is responsible for this apparent anomaly. The finer the cement the more tricalcium aluminate is available at early ages and the less effective is a given quantity of gypsum. Such lack of retardation produces a cement of high shrinkage and creep tendencies.[13] If the Type I cement was re-ground without the addition of gypsum, the resultant cement may have had a gypsum content below the optimum amount, and this would have resulted in increased creep. If the gypsum content of fine cement is increased to the optimum value, the creep tendency would decrease, probably owing to a reduction in the specific surface of the gel formed.[14]

Another aspect of the influence of fineness is its effect on the rate of hardening. For instance, old, mass concrete structures built with coarse cement show little cracking, even in a dry climate. On the other hand, where modern, more rapid hardening Portland cements of higher fineness have been employed, mass structures have shown frequent cracks of considerable intensity. This can be accounted for by the differences in shrinkage and creep characteristics of concretes made with cements of different rates of hardening.

Whether fineness is a factor per se in creep, and not only of indirect influence through its effect on the rate of hardening, remains yet to be determined experimentally.

USE OF EXPANSIVE CEMENTS

The possibility of development of creepless concrete through the use of expansive cement ought to be mentioned. Lossier [15 16] found that 0.5 per cent of expansive cement added to standard Portland cement

reduced creep by 10 per cent; 1.5 per cent of expansive cement had the effect of halving the creep. It is hoped that further tests will be made in this field: creepless concrete would be particularly interesting in the field of pre-tensioned concrete.

RESULTS OF PRESENT TESTS

The figures quoted above are only qualitative, particularly since within each type of cement there is a considerable variation in composition. The only way to a full understanding of the influence of cement composition lies in a systematic study of creep of concretes and mortars made with a variety of cements of known characteristics. Such a study has been in progress for two years but it is necessarily slow and it may be a number of years before comprehensive data will have been obtained.

In the present series of tests eleven different cements have been tested. The standard specimen used was a 2 inch diameter by 9¼ inch high cylinder made of 3 : 1 sand : cement mortar with a water-cement ratio of 0.4. The sand was pure silica Leighton Buzzard standard sand (similar to Ottawa sand); thus the possible influence of the grading and creep of the aggregate was eliminated, and the difference in creep of different mortars should be caused solely by the different cements used.

All specimens were cast in a similar manner at 20°C, the compaction being produced by means of a vibrating table. Five hours after casting the specimens were

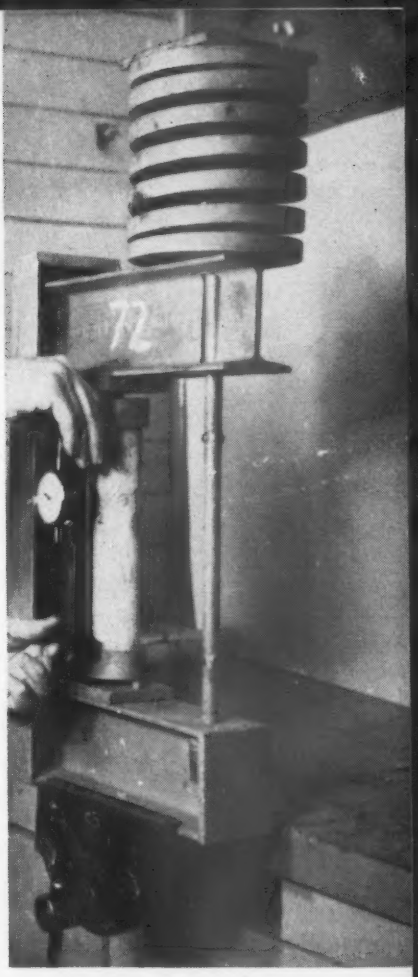

Figure 1—Apparatus for the application of sustained load: measuring the strain in the mortar specimen.

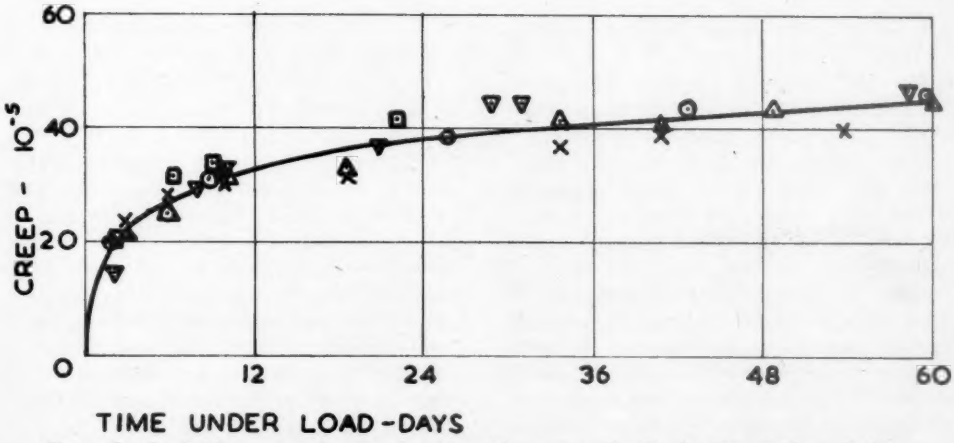

Figure 2—Creep-Time curve for mortar made with cement E4: plot for five similar cylinders.

TABLE I

Cement No.	Approximate Description		28-Day Cylinder* Strength p.s.i.	Load Applied as a Percentage of 28 Days Strength	Creep After 60 Days Under Sustained Load of 2150 p.s.i. (10^{-5})
	British	U.S. Type			
E1	Ordinary Portland	I	4080	52.5	63
E2	Ordinary Portland	—	3770	57	59.5
E3	Ordinary Portland	I	5150	41.5	42
E4	Low Heat Portland	IV	5180	41.5	45
E5	Ordinary Portland	I	3810	56.5	65
E6	Rapid-Hardening Portland	III	4470	48	59
E7	Ordinary Portland	III	5350	40	41
E8	White	—	4510	47.5	52
A1	Rapid-Hardening Portland	III	6030	35.5	44
A2	Slphate-Resisting Portland	V	4910	43.5	47
L1	High Alumina	Aluminous	6200	34.5	26.5

*Cylinder 2 inch diameter x 9¼ inch high.

capped with neat cement; at 24 hours they were stripped and immersed in water at 20°C until the age of 27 days. They were then transferred to a constant temperature (20°C) and constant humidity (95 per cent) cabinet. At the age of 28 days the specimens were loaded to a constant stress of 2150 psi in a specially designed apparatus(17), (Fig. 1) and kept continuously in the cabinet.

The strain in the mortar was measured by a Whittemore-type strain gauge engaging cast-in gauge-points on two opposite sides of the cylinder. A typical creep-time curve is shown in Fig. 2, both the creep and time being reckoned from the instant when the load was fully applied.

The cement characteristics together with the total creep after sixty days under load are given in Table I. Some trends in the magnitude of creep with the chemical composition of cement may be seen but at this stage no definite relationships can be written down. On the other hand, the inversely proportional relation of creep to strength of mortar is easily observed. This is more conveniently expressed in the form of a relationship between creep and the percentage of the ultimate load to which the specimen was subjected. For the range of percentages of the ultimate load tested, this seems to be approximately in the form of a straight line, shown in Fig. 3.

This figure includes data for cements of

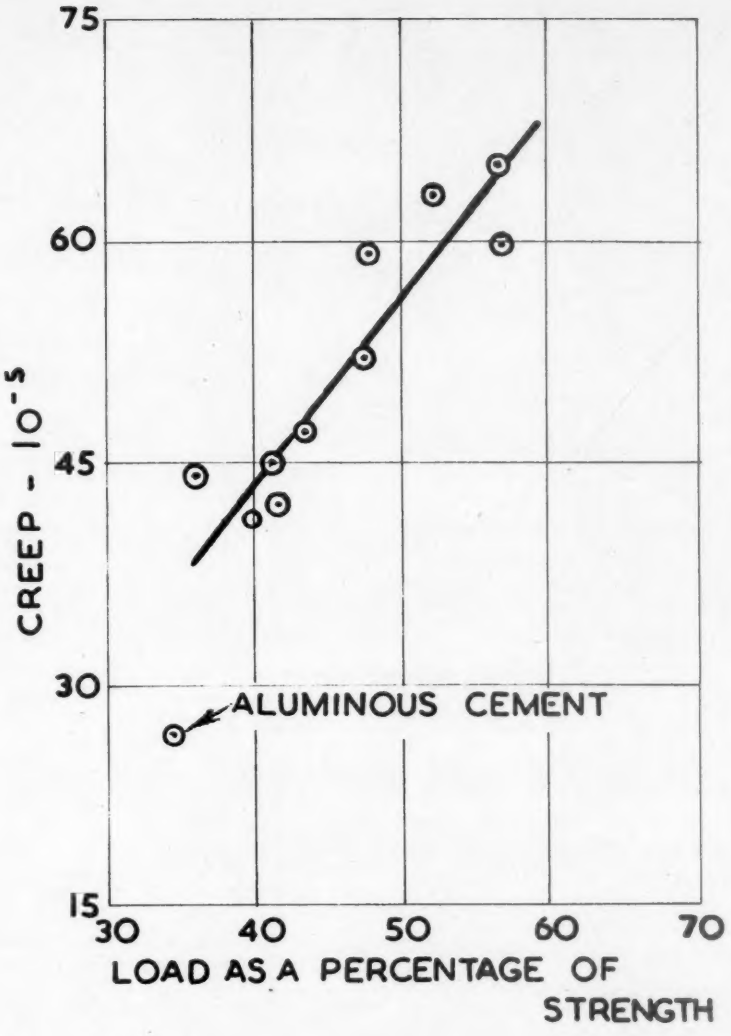

Figure 3—Creep of mortars of various strengths loaded at the age of 28 days and subjected for 60 days to a sustained load of 2150 psi.

Type I, III, IV, V, white cement and aluminous cement, all of which seem to satisfy one relationship. It must be remembered that all specimens were subjected to the same stress. There seems, therefore, to exist a strong possibility that the inherent strength of cement is the primary factor in creep of mortar and concrete made with that cement. It is well known that the magnitude of load, the age at loading, the mix proportions, the type of aggregate, and other mix properties affect creep, but the present data indicate that for any given mix and load it is the strength of cement that seems predominantly to affect the creep: stronger cement would result in lower creep. The results quoted here are expressed in terms of the mortar strength at the age of 28 days; obviously it is the strength during the whole sixty days under load that is relevant but since creep

decreases rapidly with time (Fig. 1) the strength at the time of loading will be of paramount importance. The differences in later strength development will, however, be reflected in the scatter of the results obtained.

The strength of cement has been shown to be a function of its chemical composition (2) and, through this medium, it should therefore be possible to transform the creep-strength function into the creep-composition relationship. There are, however, serious difficulties in the application of the theoretical strength-composition relationships to commercial cements manufactured in different works; these difficulties have been mentioned earlier. For this reason only the strength-creep relationship is presented. In practice, the strength of cement is more often determined than its chemical composition, and the present relation should be easily applicable. Needless to say, quantitative data would be required for any set of conditions, but, ceteris paribus, the effects of change of cement characteristics would be expected to follow a pattern similar to that of Fig. 3.

CONCLUSIONS

From the test results of various investigations, reviewed in the earlier part of the paper, it seems that, broadly, higher strength cements lead to lower creep. It is not possible, however, to infer a definite relation between creep and cement characteristics. This is mainly due to the fact that in many of the tests reported no details were given of the strengths of the concretes made with different cements, nor was full information about these cements included.

The tests performed to date by the author deal with only one load, one storage condition, one mix, one paste content and one water-cement ratio, but they show the behaviour of different cements (Portland and aluminous, but not Blast Furnace and Pozzolana) under this one set of conditions. It seems that for these conditions creep is inversely proportional to the strength of cement, i.e. other things being equal, the stronger the cement the lower the creep.

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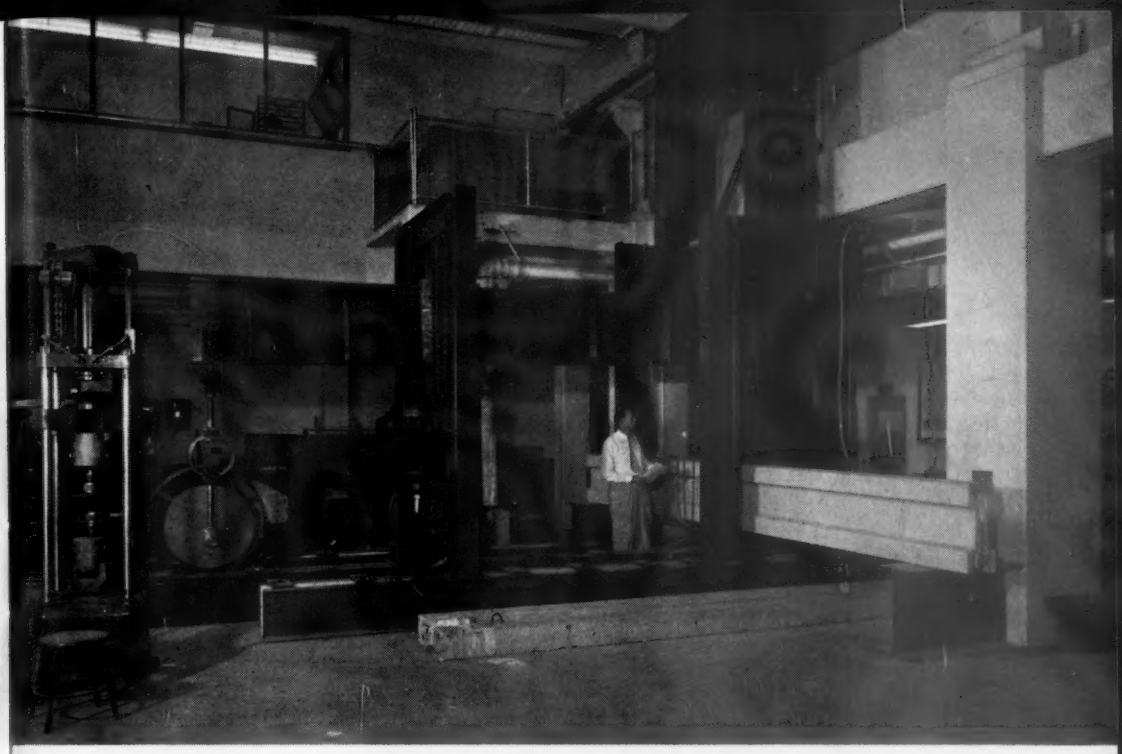

Figure 1—Fatigue machine used in the tests

Composite Prestressed Concrete Beams Under Repetitive Loading

By A. M. Ozell* and J. F. Diniz**

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THE purpose of these tests was to study the behavior of composite pretensioned prestressed beams under repetitive loading. Previous tests[1] indicated that under static loading natural bond between the two concretes, even on smooth contact surfaces, was sufficient to develop full composite action without any mechanical tie between the prestressed and cast-in-place portions of composite beams. It was believed, however, that the use of a few shear ties would insure this action under repeated design loads and over-loads.

Four 20 ft. long beams were tested. The plane of contact of the prestressed portion was trowel smooth finished. Only four No. 4 shear ties spaced at 6 in. at each end of the composite beam were provided to pre-

[1]Ozell, A. M., "Behavior of Simple-Span Composite Prestressed Concrete Beams," Journal Prestressed Concrete Institute, June, 1957, p. 18.

vent the separation of the concretes.

A fatigue machine of the constant-deflection type was used (Fig. 1). This machine was designed and built in the testing laboratory of the Civil Engineering Department of the University of Florida at Gainesville. Essentially it consists of a rocker arm connected to an adjustable eccentric driven by an electric motor. The rocker arm is connected to the eccentric through a dynamometer which measures the load applied to the specimen.

Description of Specimens: Each of the composite beams consisted of a rectangular prestressed portion 8 x 10 in. and pretensioned with three 7/16 in. seven-wire strands (initial tension of 18,900 lbs. per strand) placed 2 in. from the bottom of the beam. The strands were straight and bonded along the entire length of the beam.

Two groups of beams were tested. In one group, the cast-in-place portion was 8 x 10 in. resulting in a rectangular composite section 8 x 20 in. In the other group the cast-in-place portion was a 3 x 34 in. flange resulting in a tee-section for the composite beam. The reinforcing for the flange consisted of No. 4 bars spaced at 5 in. longitudinally and 6 in. transversely. Details of specimens are given in Fig. 2.

The prestressed portions of the composite beams were wet-burlap cured for four days before stress release at which time the cylinder strength was in excess of 4,000 psi. Mix design per cubic yard of concrete was as follows:

8.5 bags type III cement
27 gallons water
8½ quarts Pozzolith
1094 lb. Tavares sand (Fineness modulus: 2.30, specific gravity: 2.63)
1880 lb. Brooksville stone (SRD type 9; specific gravity: 2.52)

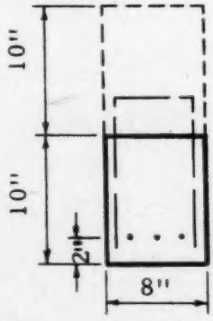
The cast-in-place portion was cast with a 4,000 psi concrete obtained from a local ready-mix plant and was vibrated mechanically and wet-burlap cured for five days. The contact surface was not grouted but kept wet for several hours before casting.

Fatigue Loading: All the beams were loaded at quarter points through a 27 W F 94 loading beam, as shown in Fig. 3. The total superimposed fatigue loading for beam R 1 varied between minimum load of 1880 to maximum load of 16530 lb. and for beam R 2 from 12,400 to 22,400 lb. Loading for beam T 1 varied between 2,000 to 11,000 lb. and for beam T 2 from 2,000 to 10,000 lb.

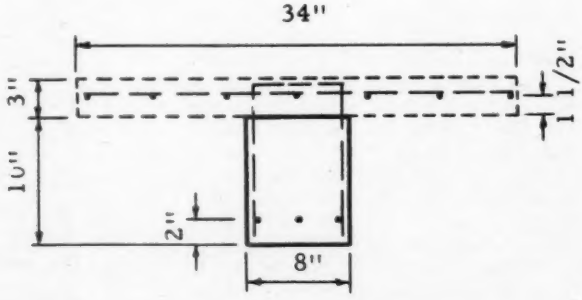
The calculated stresses in the composite beams, including the effect of shrinkage (see Appendix for stress calculations caused by differential shrinkage) and assuming 20 per cent stress loss are given below:

Specimen R 1	
under minimum load,	
stress at bottom fiber	— 550 psi
stress at top fiber	— 297 psi
under maximum load,	
stress at bottom fiber	+ 247 psi
stress at top fiber	—1094 psi
Specimen R 2	
under minimum load,	
stress at bottom fiber	+ 24 psi
stress at top fiber	— 869 psi
under maximum load,	
stress at bottom fiber	—1414 psi
stress at top fiber	+ 567 psi

Figure 2—Details of the Specimens



R-1 and R-2



T-1 and T-2

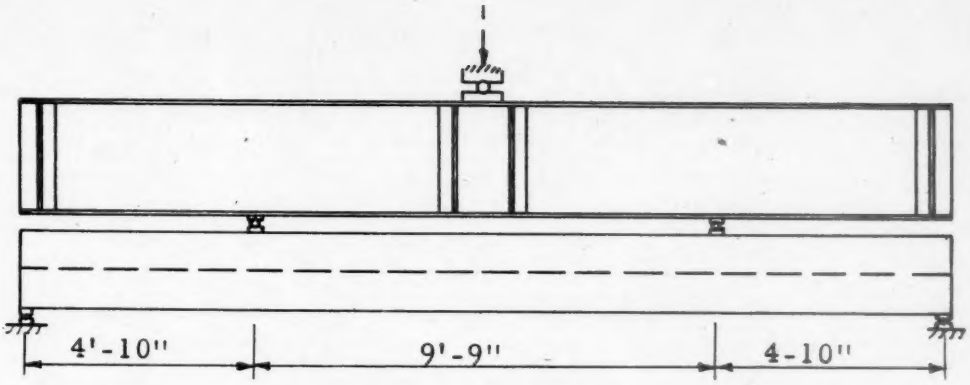


Figure 3—Loading Diagram

Specimen T 1	
under minimum load,	
stress at bottom fiber	— 196 psi
stress at top fiber	— 142 psi
under maximum load,	
stress at bottom fiber	+ 667 psi
stress at top fiber	— 578 psi
Specimen T 2	
under minimum load,	
stress at bottom fiber	— 196 psi
stress at top fiber	— 142 psi
under maximum load,	
stress at bottom fiber	+ 572 psi
stress at top fiber	— 530 psi

On the calculation of the above stresses it was noted that the coefficient of differential shrinkage plays an extremely important part in composite beam stresses. The magnitude of this coefficient must be properly chosen for an accurate analysis or design. On the other hand, the respective values of the moduli of elasticity of the two concretes has little effect relatively, on the stresses introduced in a composite beam as a result of differential shrinkage.

BEHAVIOR OF SPECIMENS DURING TESTS

Beams of Group R: In specimen R 1 the first six cracks appeared between 0 and 1000 repetitions of the load, shown as numbers 1 through 6 in Fig. 4. The other cracks developed at 65,000, 75,000 and 78,000 cycles. Cracks 7 through 10 appeared at 65,000 cycles and crack 8 was the extension of crack 1 into the cast-in-place portion of the beam. Crack 12 was similarly the extension of crack 5. Up to the time when testing was discontinued at 452,000 cycles no additional cracks appeared.

At 65,000 cycles there was an apparent permanent set. The eccentric was then adjusted to maintain the required load. Because the dial gage used to measure the deflection had accidentally come loose from its initial position, it was not possible to measure accurately the permanent set during the first 65,000 cycles of load.

At 452,000 cycles the test was discontinued since contact between the loading beam and the specimen could not be maintained in spite of frequent adjustments of the eccentric, indicating excessive permanent set. Although this was considered to be evidence of failure of the specimen, no failure of the strands was observed upon breaking into the specimens and examining them. There was some evidence of bond failure between the strands and the concrete.

The pattern of cracks for beam R 2 is shown in Fig. 4. Cracks 1 through 8 appeared at the first application of the load. At 10,000 cycles crack 9 appeared and crack 7 progressed into the cast-in-place portion of the beam. At 400,000, crack 10 appeared and at 550,000 the group of cracks 11 and 12 developed. Failure occurred at 580,000 cycles.

In beam R 2, permanent set could not be observed because of the faulty attachment of the dial gage.

When the test was discontinued at 580,000 cycles and the beam was broken into, it was found that two out of three strands had failed; one wire in one and six wires in another strand had broken as shown in Fig. 5.

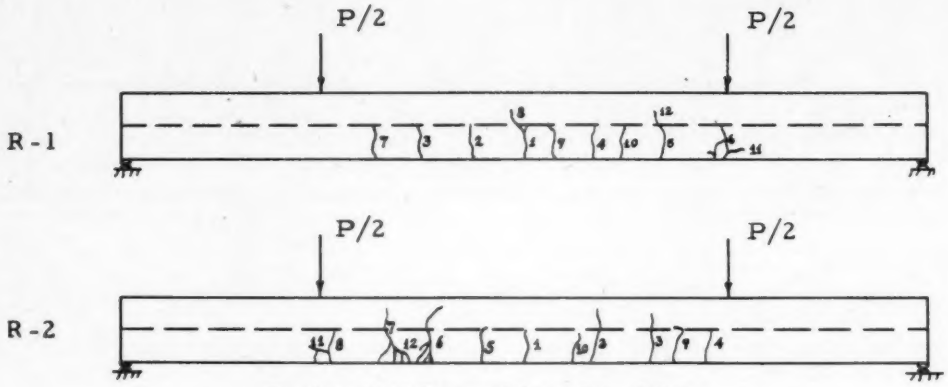


Figure 4—Crack Pattern of Beams R-1 and R-2

Beams of Group T: The testing of the group T beams was conducted with more instrumentation than with the R beams. The accuracy of the loading mechanism was checked by comparing the load indicated by the dynamometer ***versus*** reaction values at each support, by placing the beams on loading pads. Each loading pad consisted of an 8 in. x 8 in. x 1 in. steel plate. Two SR-4 electric strain gages, type A-1, fastened to the bottom of each plate measured the strain caused by bending of the plate under the applied load or reaction. The pads were calibrated and found to have an accuracy of 3 per cent within the test-load limits. The results showed that the dynamometer-load readings were within 5 per cent of the measured loads. Also, strains in the prestressed portion of the composite beam were measured by a mechanical strain gage and the results were in close agreement with the calculated strains indicating the accuracy of the loading of the fatigue machine.

The pattern of cracks for specimen T 1 is shown in Fig. 6. At 800,000 cycles it was necessary to change the throw of the eccentric so that proper contact and the application of the test load could be maintained. At 865,000 cycles the test was discontinued. The beam was broken into and no strands

Figure 5
Broken strand
in Beam R-2

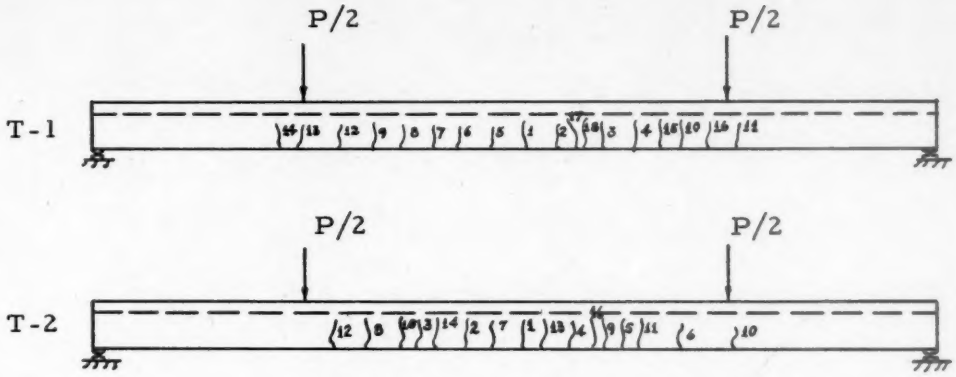


Figure 6—Crack Pattern of Beams T-1 and T-2

were found to be damaged. There was evidence of bond failure within the portion of the beam between the loading points. Both the strands and the grooves left by them in the concrete were smooth.

Fig. 6 shows the pattern of cracks for specimen T 2. Cracks 1 to 7 appeared at the first application of the test load. The other cracks appeared as follows: 8 at 40,000, 9 at 50,000, 10 at 100,000, 11 at 116,000, 12 and 13 at 220,000, 14 at 425,000, 15 and 16 at 650,000 cycles.

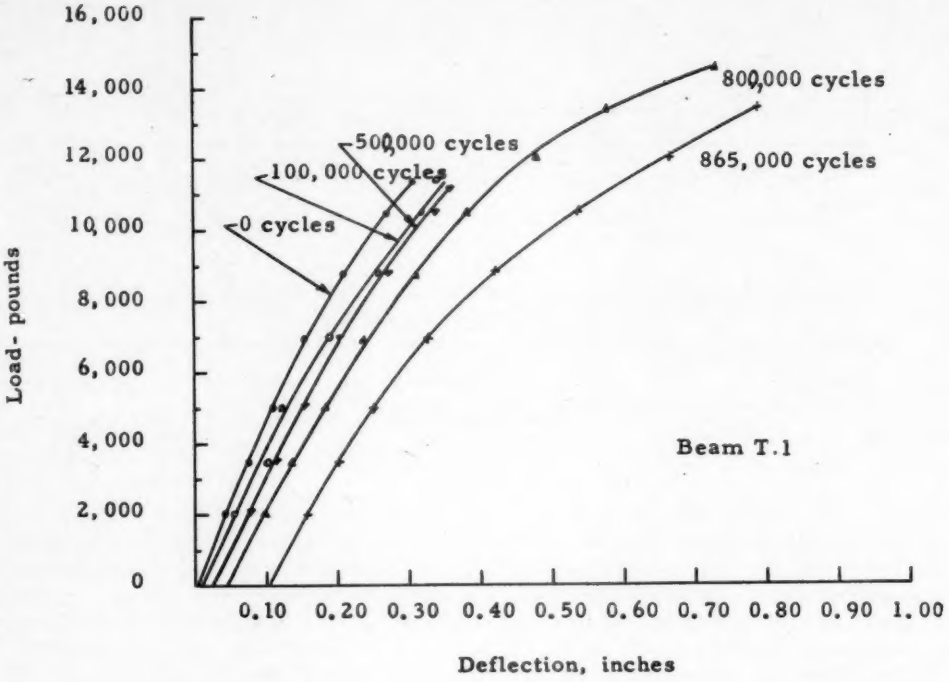
At 650,000 cycles the throw of the eccentric was changed and at 860,000 cycles the test was stopped. Upon breaking into the beam no strand failure was found but all the strands showed bond failure. It is interesting to note that there was no excessive amount of rust on two of the strands, one of which was spliced at about 4 ft. from one end of the beam.

Load-deflection relationship of beams T 1 and T 2 as affected by fatigue is presented in Fig. 7 while Fig. 8 shows the permanent set *versus* cycles of loading diagram for these beams.

At the time of testing the average cylinder strengths of the cast-in-place and the prestressed concrete portions of the composite beams were 5650 psi and 6350 psi, respectively.

CONCLUSIONS

1. None of the composite beams tested showed any sign of failure along the plane of contact. On three specimens the bond between the strand and the concrete failed and in one specimen failure was by fracture of the wires in two of the strands.
2. The natural bond between the two concretes and the presence of four shear ties at each end of the beams provided a composite action strong enough to withstand repetitive over-loads.
3. Based on zero bottom-fiber stress as design load, composite beams having rectangular cross sections failed at an average maximum load of 1.7 times the design live-load at about 500,000 cycles and the tee-beams failed at about 860,000 cycles under an average maximum repetitive load of 2.6 times the design live load.
4. The coefficient of differential shrinkage plays an extremely important part in composite-beam stresses. The magnitude of this coefficient must be properly chosen for an accurate analysis or design. For beams used in these tests the difference in the live-load safety factor varied almost 100 per cent when based on two differential shrinkage coefficients of 0.0002 and 0.0004.
5. It seems important that more study be devoted to the proper evaluation of the coefficient of differential shrinkage for various strength concretes.
6. The respective values of the moduli of elasticity of the two concretes has little effect on the stresses introduced in a composite beam as a result of differential shrinkage.



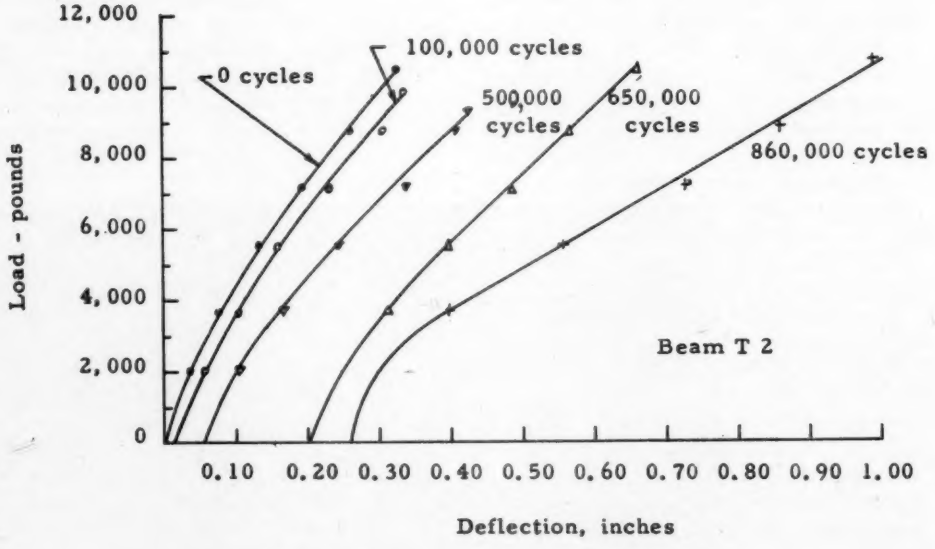


Figure 7—Load-Deflection Diagram of Beams T-1 and T-2

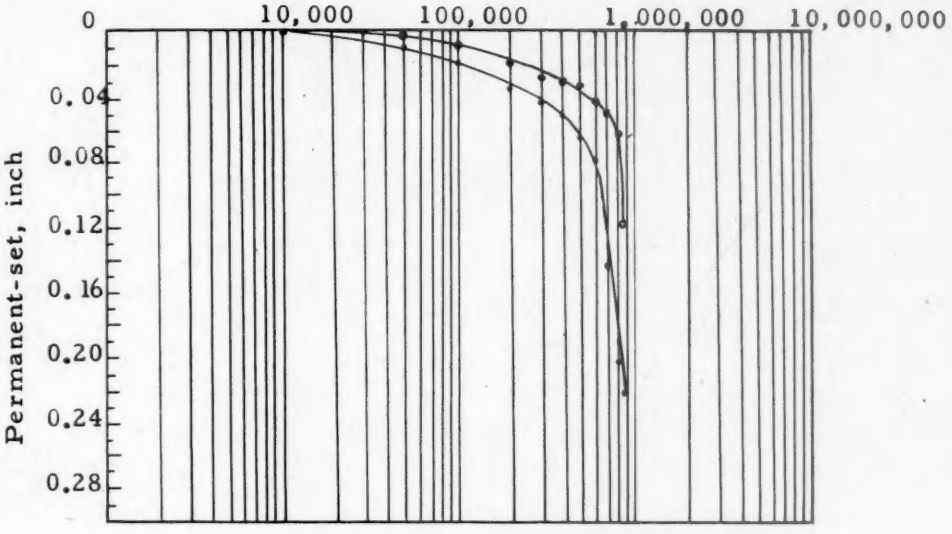


Figure 8—Permanent Set vs. Cycles of Loading Diagram

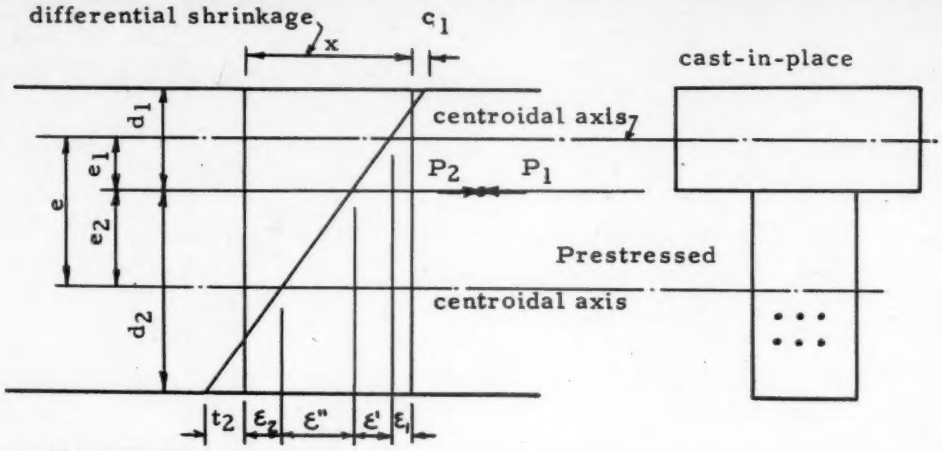
Appendix

SHRINKAGE STRESSES IN COMPOSITE BEAMS[1]

Notation:

x = Differential shrinkage coefficient
d_1 = Depth of cast-in-place portion
d_2 = Depth of prestressed concrete portion
e = Distance between centers of gravity of the two concretes
e_1, e_2 = Distance from line of action of mutual force to centers of gravity of cast-in-place and prestressed portion, respectively
c_1 = Compressive strain in the top fiber of the cast-in-place concrete
ϵ_1 = Strain at the centroidal axis of the cast-in-place concrete due to direct load
ϵ' = Strain at the bottom fiber of the cast-in-place concrete due to bending
t_2 = Tensile strain in the bottom fiber of the prestressed portion
ϵ_2 = Strain at the centroidal axis of the prestressed portion due to direct load
ϵ'' = Strain at the top fiber of the prestressed portion due to bending
A_1, A_2 = Area of cast-in-place and prestressed portions, respectively
E_1, E_2 = Modulus of elasticity of cast-in-place and prestressed portions, respectively

1Evans, R. H. and Parker, A. S., Journal American Concrete Institute. Vol. 26, May 1955, pp. 861-878.



Equilibrium condition 1:

The resultant tension in the cast-in-place concrete must equal the resultant compression in the prestressed concrete:

$P_1 = P_2$ which can also be expressed in terms of strains at the centroidal axis of the respective concrete as

(1) $\epsilon_1 E_1 A_1 = \epsilon_2 E_2 A_2$

(2a) $\epsilon_1 = \dfrac{E_2 A_2}{E_1 A_1} \epsilon_2$ or

(2b) $\epsilon_2 = \dfrac{E_1 A_1}{E_2 A_2} \epsilon_1$

Equilibrium condition 2:

The resultant tension and compression cannot form a couple as there is no external bending moment. They must act in the same line. The mutual force between the concretes due to differential shrinkage acts as an eccentric load on each section with respective eccentricities of $e_1 e_2$. These cause moments of $P_1 e_1$ and $P_2 e_2$ and the resulting stresses could be expressed in terms of strains in the usual flexural formula:

$$\text{Stress} = \text{Unit Strain} \times E = \frac{Pe\,y}{I}$$

Hence the stress in the bottom fiber of the cast-in-place portion can be expressed as:

$$\epsilon' E_1 = \frac{P_1 e_1 (e_1)}{I_1} = \frac{\epsilon_1 E_1 A_1 e_1^2}{I_1}, \text{ or}$$

(3a) $\epsilon' = \dfrac{\epsilon_1 A_1 e_1^2}{I_1}$

Similarly the stress in the top fiber of the prestressed portion can be expressed as:

(3b) $\epsilon'' = \dfrac{\epsilon_2 A_2 e_2^2}{I_2}$

Substituting the values of ϵ_1 and ϵ_2 in Eq. (1), we have:

(4) $\dfrac{\epsilon' I_1 e_2^2}{\epsilon'' I_2 e_1^2} = \dfrac{E_2}{E_1}$

From the strain diagram, we find that

$\dfrac{\epsilon'}{e_1} = \dfrac{\epsilon''}{e_2}$ or $\epsilon' = \dfrac{e_1}{e_2} \epsilon''$ when substituted in Eq. (4):

(5) $$\frac{e_1}{e_2} = \frac{E_1 I^1}{E_2 I_2}$$

But $x = \epsilon_2 + \epsilon'' + \epsilon' + \epsilon_1$ and making the proper substitutions:

$$x = \frac{\epsilon_2 A_2 e_2^2}{I_2} + \frac{\epsilon_1 A_1 e_1^2}{I_1} + \frac{E_2 A_2 \epsilon_2}{E_1 A_1} + \frac{E_1 A_1}{E_2 A_2} \epsilon_1$$

and substituting Eq. (2a) in the above expression, we have:

(6) $$x = \epsilon_2 \left(1 + \frac{A_2 e_2^2}{I_2} + \frac{E_2 A_2 e_1^2}{E_1 I_1} + \frac{E_2 A_2}{E_1 A_1}\right)$$, and similarly

(7) $$x = \epsilon_1 \left(1 + \frac{A_1 e_1^2}{I_1} + \frac{E_1 A_1 e_2^2}{E_2 I_2} + \frac{E_1 A_1}{E_2 A_2}\right)$$

The strains along the centroidal axis of the two portions of the composite beam could be calculated by using Eq. (6) and (7) provided the magnitude of x, the differential shrinkage coefficient, as well as the moduli of elasticity of the two concretes are known. For these tests the following values were used:

$x = 2 \times 10^{-4}$ in/in

$E_1 = 4 \times 10^6$ psi

$E_2 = 4.5 \times 10^6$ psi

For the rectangular composite section the unit strains along the centroidal axis of the two concretes were found to be:

$\epsilon_1 = 2.65 \times 10^{-5}$ in/in and $\epsilon_2 = 2.35 \times 10^{-5}$ in/in

By proper substitution into Eq. (3a) and (3b) values of ϵ' and ϵ'' were found to be:

$\epsilon' = 7.95 \times 10^{-5}$ in/in and $\epsilon'' = 7.05 \times 10^{-5}$ in/in.

From the strain diagram the top and bottom fiber unit strains for the two concretes could be expressed as follows:

Cast-in-place portion:

Top fiber strain: $\frac{c_1 + \epsilon_1}{d_1 e_1} = \frac{\epsilon' + \epsilon''}{e} \therefore c_1 = \frac{d_1 - e_1}{e}(\epsilon' + \epsilon'') - \epsilon_1$

Bottom fiber strain: $\epsilon_1 + \epsilon'$

Prestressed portion:

Top fiber strain: $\epsilon_2 + \epsilon''$

Bottom fiber strain: $\frac{t_2 + \epsilon_2}{d_2 - e_2} = \frac{\epsilon' + \epsilon''}{e} \therefore t_2 = \frac{d_2 - e_2}{e}(\epsilon' + \epsilon'') - \epsilon_2$

From the above strain values the following stresses were calculated for the rectangular composite beam:

Cast-in-place portion:

Top fiber stress $= 4.85 \times 10^{-5}$ in/in x $E_1 = -194$ psi (compression)

Bottom fiber stress $= 10.60 \times 10^{-5}$ in/in x $E_2 = +423$ psi (tension)

Prestressed portion:

Top fiber stress $= 9.40 \times 10^{-5}$ in/in x $E_2 = -423$ psi

Bottom fiber stress $= 5.15 \times 10^{-5}$ in/in x $E_2 = +232$ psi

For the composite beam having the tee-section the following strains were calculated:

$\epsilon_1 = 2.34 \times 10^{-5}$ in/in and $\epsilon_2 = 2.65 \times 10^{-5}$ in/in

$\epsilon' = 7.05 \times 10^{-5}$ in/in and $\epsilon'' = 7.95 \times 10^{-5}$ in/in

Using the above strain values the following stresses were determined:

Cast-in-place portion:

Top fiber stress $= 1.12 \times 10^{-5} \times E_1 = -45$ psi

Bottom fiber stress $= 9.39 \times 10^{-5} \times E_1 = +376$ psi

Prestressed portion:

Top fiber stress $= 10.60 \times 10^{-5} \times E_2 = -476$ psi

Bottom fiber stress $= 8.88 \times 10^{-5} \times E_2 = +400$ psi

JOINT ASCE-ACI REPORT ON

PRESTRESSED CONCRETE

Prepared by COMMITTEE 323

This Report is herein published with the approval of the AMERICAN CONCRETE INSTITUTE and the AMERICAN SOCIETY OF CIVIL ENGINEERS.

No reprints or individual copies of the Report can be published by the PRESTRESSED CONCRETE INSTITUTE. Separate reprints are available only from the AMERICAN CONCRETE INSTITUTE.

Discussion of all aspects of this tentative Report should be directed to the AMERICAN CONCRETE INSTITUTE not later than April 1, 1958.

This Report is being published by the PRESTRESSED CONCRETE INSTITUTE in order to acquaint its membership with this tentative Report, which sometime in the future will be adopted and used throughout the United States as a standard for the design and construction with prestressed concrete.

A. M. OZELL,
Editor
PCI POURNAL

SYNOPSIS

A guide to design and construction of safe, serviceable, linear structural members prestressed with high strength steel. Emphasis is on flexural members —beams, girders, and slabs. Most of the recommendations are applicable to both buildings and bridges. Design chapter treats: loading; allowable stress; prestress loss; flexure and shear; bond and anchorage; composite construction; continuity; end blocks; fire resistance; and cover and spacing of prestressing steel. Concrete, grout, prestressing steel, anchorages, and splices are covered in the section on materials. Construction section includes: transportation, placing, and curing of concrete; forms, shoring, and falsework; placement of prestressing steel and application of the prestressing force; grouting; and handling and erection.

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CHAPTER 1—INTRODUCTION

101—OBJECTIVE

The objective of this report is to recommend those practices in design and construction which will result in prestressed concrete structures that are comparable both in safety and in serviceability to constructions in other materials now commonly used.

This report constitutes a recommended practice, not a building code or specification. Since it was not written as a code, its use or interpretation as one will not serve the best interests of either the public or the engineering profession. Recommendations contained in the report are presented solely for the guidance and information of professional engineers. Safety and economy of structures in prestressed concrete will depend as much on the intelligence and integrity of engineers preparing the design and supervising or carrying out the construction as on the degree to which these recommendations are followed.

102—SCOPE

102.1—Linear prestressing

This report is confined in scope to linear structural members involving prestressing with high strength steel; circularly prestressed members such as tanks or pipes are not covered. These types of construction have been excluded for two reasons. They have been designed and constructed in this country for a great number of years and procedures have been developed on the basis of research and experience which have proved successful in practice. Design and construction of tanks and pipes in prestressed concrete are confined to a relatively small group of specialists and are not likely to be attempted by persons outside that group. For these reasons there seems to be no immediate need for recommendations regarding circularly prestressed structures.

102.2—Flexural members

For the most part, recommendations in this report relate to flexural members—beams, girders, and slabs. Other structural forms, such as columns, ties, arches, shells, trusses, pavements, etc., are treated only briefly or not at all. In some of these cases, such as columns or ties, the principles involved in design are essentially simple and no need was felt to include them in this report. In other cases, insufficient information was available either from research or experience to permit recommendations to be made at this time. This lack of information is due in some instances to the complexity of the type of structure involved and in others to the infrequency of its use in this country.

102.3—Buildings and bridges

These recommendations are intended to apply to both buildings and bridges. The form and nature of this report are such that almost all recommendations made apply without differentiation to both types of structures. Where this is not the case, separate recommendations are made for buildings and bridges.

103—ACCEPTANCE TESTS

It is recognized by the committee that unusual types of construction, design, or materials may be used in such a manner that these recommendations are not applicable or may not have been complied with. Such structures may be adequate for the purpose intended. In these cases it is recommended that tests be made to verify design.

104—NOTATION

104.1—General

Symbols are assembled into sections pertaining to groups of associated terms. The list comprises only the symbols in this report. No attempt is made to present a complete notation for design of prestressed concrete.

104.2—Dimensions and cross-sectional constants

A_b = bearing area of anchor plate of post-tensioning steel

A_c = maximum area of the portion of the anchorage surface that is geometrically similar to and concentric with the area of the bearing plate of post-tensioning steel

A_s = area of main prestressing tensile steel

A_s' = area of conventional tensile steel

A_{sf} = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange

A_{sw} = steel area required to develop the ultimate compressive strength of the web of a flanged section

A_v = area of web reinforcement placed perpendicular to the axis of the member

b = width of flange of a flanged member or width of a rectangular member

b' = width of web of a flanged member

d = distance from extreme compressive fiber to centroid of the prestressing force

I = moment of inertia about the centroid of the cross section

j = ratio of distance between centroid of compression and centroid of tension to the depth d

p = A_s/bd; ratio of prestressing steel

p' = ratio of conventional reinforcement

pf_s'/f_c' = percentage index

s = longitudinal spacing of web reinforcement

t = average thickness of the flange of a flanged member

Q = statical moment of cross section area, above or below the level being investigated for shear, about the centroid

104.3—Loads

D = effect of dead load

L = effect of design live load including impact, where applicable

W = effect of wind load, or earthquake load, or traction forces

V_c = shear carried by concrete

104.4—Stresses and strains

E_c = flexural modulus of elasticity of concrete

E_s = modulus of elasticity of prestressing steel

f_c' = compressive strength of concrete at 28 days

f'_{ci} = compressive strength of concrete at time of initial prestress

f_{cp} = permissible compressive concrete stress on bearing area under anchor plate of post-tensioning steel

f_s' = ultimate strength of prestressing steel

f_{se} = effective steel prestress after losses

f_{si} = initial stress in prestressing steel after seating of the anchorage

f_{su} = stress in prestressing steel at ultimate load

f_{sy} = nominal yield point stress of prestressing steel

f_t' = flexural tensile strength of concrete; modulus of rupture

f_y' = yield point stress of conventional reinforcing steel

k_2 = ratio of distance between extreme compressive fiber and center of compression to depth to neutral axis

k_1k_3 = ratio of average compressive concrete stress to cylinder strength, f_c'

n = ratio of E_s/E_c

u_d = strain in concrete due to creep

u_e = strain in concrete due to elastic shortening

u_s = strain in concrete due to shrinkage

v = shearing stress

δ_1 = ratio of loss in steel stress due to relaxation of prestressing steel

δ_2 = ratio of loss in steel stress due to friction during prestressing

104.5—Friction during prestressing

e = base of Naperian logarithms
K = friction wobble coefficient per ft of prestressing steel
T_o = steel stress at jacking end
T_x = steel stress at any point x
μ = friction curvature coefficient
α = total angular change of prestressing steel profile in radians from jacking end to point x
L = length of prestressing steel element from jacking end to point x

CHAPTER 2—DESIGN

201—GENERAL CONSIDERATIONS

201.1—Purpose

The purpose of design is to define a structure that can be constructed economically, that will perform satisfactorily under service conditions, and will have an adequate ultimate load capacity.

201.2—Mode of failure

Ultimate strength should be governed preferably by elongation of the prestressing steel rather than by shear, bond, or concrete compression.

201.3—Design theory

The elastic theory should be used at design loads with internal stresses limited to recommended values. The ultimate strength theory also should be applied to insure that ultimate capacity provides the recommended load factors.

202—SPECIAL CONSIDERATIONS

202.1—Loading conditions

Consideration should be given to all critical loading conditions in design including those that occur during fabrication, handling, transportation, erection, and construction.

202.2—Deflections

Camber and deflection may be design limitations and should be investigated for both short and long time effects.

202.3—Length changes

Length changes of concrete due to prestress and other causes should be investigated for both short and long time effects.

202.4—Reversal of loading effects

Where reversal of moment or shear may occur it should be considered in the design.

202.5—Buckling

General buckling due to prestressing can occur only over the length between points of contact of the prestressing steel with the concrete.

General buckling of an entire member or local buckling of thin webs and flanges under external loads may occur in prestressed concrete as in members made of other materials and should be provided for in design.

203—ASSUMPTIONS

203.1—Basic assumptions

The following assumptions may be made for design purposes:

a. Strains vary linearly over the depth of the member throughout the entire load range.
b. Before cracking, stress is linearly proportional to strain.
c. After cracking, tension in the concrete is neglected.

203.2—Modulus of elasticity

When accurate values for modulus of elasticity are not available, the following values may be used as a guide:

a. Flexural modulus of elasticity of concrete E_c, in psi, may be assumed to be 1,800,000 plus 500 times the cylinder strength at the age considered. Actual values may vary as much as 25 percent from those given by the foregoing expression. This expression is not applicable to lightweight concrete, for which E_c should be determined by test.

b. Modulus of elasticity of steel, in psi, may be assumed to be 29,000,000 for cold drawn wire, 27,000,000 for 7-wire strand, 25,000,000 for strand with more than 7 wires, and 27,000,000 for alloy steel bars.

203.3—Deflections

Deflection or camber under short time loading may be computed using values of E_c obtained as described in Section 203.2.a.

Deflection associated with dead load, prestress, and live loads sustained for a long time may be computed on the assumption that the corresponding concrete strains are increased as a result of creep. The increase in strain may vary from 100 percent of the elastic strain in very humid atmosphere to 300 percent of the elastic strain in very dry atmosphere. These values may not pertain to concrete made with lightweight aggregates.

204—LOADING STAGES

204.1—Loading

Loading stages listed in the following sections should be investigated. No attempt is made to list all significant loading stages that may occur. Stages listed are those that normally affect the design.

204.2—Initial prestress

Prestressing forces are applied in prearranged sequence and sometimes in stages. If prestressing forces are not counteracted by the effect of the dead load of the member, or if the stressing operation is accompanied by temporary eccentricities, concrete stresses should be investigated.

204.3—Initial prestress plus dead load of member

For determination of concrete stresses at this stage, losses in prestress are those which occur during and immediately after transfer of prestress.

204.4—Transportation and erection

Support conditions for precast members during transportation and erection may differ from those during service loads. Handling stresses should be included together with prestress and dead load. Losses in initial prestress up to time of handling should be considered.

204.5—Design load

This stage includes stress due to effective prestress after losses, dead loads, and maximum specified live load.

204.6—Cracking load

Complete freedom from cracking may or may not be necessary at any particular loading stage. Type and function of the structure and type, frequency, and magnitude of live loads should be considered.

204.7—Temporary overload

This stage refers to any large live load in excess of design load, which is of short-time duration and expected to occur infrequently during life of structure. For such a load, stresses may exceed those recommended for design load but elastic recovery must be assured.

204.8—Ultimate load

Ultimate load is that load which applied statically in a single application causes failure. Such a large load would never intentionally be placed on the structure, but it is used as a measure of safety. In statically determinate structures, failure will occur at a single cross section. In statically indeterminate structures, the load which causes moment in one section to reach its ultimate value may not be sufficient to cause failure of the structure because of moment redistribution. Since it is not always possible to predict that full redistribution will take place in accordance with limit design, it is suggested for the time being that moments be determined by elastic analysis.

205—LOAD FACTORS

205.1—General

A load factor is a multiple of the design loads used to insure safety of the structure.

205.2—Cracking load factors

If cracking of concrete is undesirable, load factors for cracking load should be chosen to reflect the greatest load that can be expected during life of structure.

Formation of a crack under temporary overload may not be objectionable. If reopening such a crack under subsequent design load is objectionable it may be avoided by proper choice of concrete stress permitted for cracking load.

205.3—Ultimate load factors

The ultimate load capacity should be computed since stresses are not linearly proportional to external forces and moments throughout the entire load range. For the present, it is recommended that moments, shears, and thrusts produced by external loads and prestressing forces be investigated by elastic analysis.

The load factors recommended are believed to be consistent with current viewpoints. It may be desirable to modify or expand the load factor formulas to fit special conditions that may occur in unusual structures, extremely long spans, or for unique loadings. Deviations from the recommended values should be substantiated by suitable investigations.

205.3.1—*Buildings*

For the present, to correlate prestressed concrete with reinforced concrete practice in current use, the committee recommends that ultimate load capacity be investigated to insure meeting the following requirements:

$$1.2\,D + 2.4\,L$$

or $$1.8\,(D + L)$$

or $$1.2\,D + 2.4\,L + 0.6\,W$$

or $$1.2\,D + 0.6\,L + 2.4\,W$$

or $$1.8\,(D + L + \tfrac{1}{2}\,W)$$

or $$1.8\,(D + \tfrac{1}{2}\,L + W)$$

whichever is greater.

205.3.2—*Highway bridges*

The following load factors for highway bridges are recommended by a subgroup appointed by American Association of State Highway Officials, Committee on Bridges and Structures.*

$$1.5\,D + 2.5\,L$$

The committee is not prepared at this time to make recommendations for load factors involving the effect of lateral loads on bridges.

205.3.3—*Railway bridges*

Ultimate load factors for railway bridges are currently being studied by the American Railway Engineering Association. The committee is not prepared to recommend such factors at this time.

206—REPETITIVE LOADS

206.1—General

Ultimate strength of concrete or steel subjected to repetitive loading may be less than static strength because of the phenomenon of fatigue. Full importance of fatigue in prestressed concrete members has not yet been determined. Fatigue failure may occur in concrete, steel, anchorages, splices, or bond.

*These load factors are considered adequate for spans of moderate length, simply supported. For exceptionally long spans and for continuous members, special investigation to consider a possible increase in load factors is recommended.

206.2—Concrete

Fatigue strength of concrete in both tension and compression depends on magnitude of stress, range of stress variation, and number of loading cycles. Since high stresses and stress ranges are common, fatigue should be considered when repetition of loading cycles may occur.

Fatigue failure is unlikely if the allowable stresses of Section 207.3.2 are not exceeded and there is no reversal of stress. If a large number of overloads are anticipated a reduction in the safety factor may occur.

206.3—Prestressing steel

Fatigue strength of prestressing steel depends on magnitude and range of stress, and number of cycles of loading. Minimum stress is the effective prestress. Maximum stress and range of stress depend on magnitude of live loads or overloads that may be repeated. Range of stress under service loads will usually be small unless concrete is cracked. Cracking may occur if tension is permitted in concrete. Fatigue failure of steel should be considered in such cases, especially when a high percentage of ultimate strength is used for prestress.

Devices for splicing steel may contain strain concentrations that lower fatigue strength. Consideration should be given to fatigue whenever splices are used.

206.4—Anchorages

If steel is fully bonded, no difficulty should be expected in the anchorage or end bearing as the result of repetitive loads. With unbonded steel, fluctuations in stress due to repeated service loads or overloads are transmitted directly to anchorages and fatigue strength of the anchorage will require special consideration.

206.5—Bond

Failure of bond under repetitive loading is unlikely unless the member is cracked under design loads or a significant number of repetitions of overload. High bond stresses adjacent to cracks may be a source of progressive failure under repeated loads.

206.6—Shear and diagonal tension

Since inclined cracks may form under repetitive loading at appreciably smaller stresses than under static loading, web reinforcement should always be provided in members subjected to repetitive loading.

206.7—Design recommendations

Fatigue should not result in a reduction of strength if the following recommendations are observed. When the recommendations cannot be followed, fatigue strength of all elements comprising the prestressed member should be considered.

a. Flexural compressive concrete stress should not exceed $0.4f_c'$ under either design load or an overload that may be repeated many times.

b. Tension should not be permitted in concrete at the critical cross section under either design load or overloads that may be repeated a large number of times.

c. Reversal of stress should not occur under repeated loads.

d. Prestressing steel should be bonded.

e. Web reinforcement should be provided.

207—ALLOWABLE STEEL AND CONCRETE STRESSES

207.1—Prestressing steel

207.1.1—*Temporary stresses*

Under normal design loads stress in prestressing steel will almost always be less than stress at initial prestress. Stress at the anchorage immediately after seating has been effected should not exceed $0.70f_s'$ for material having stress-strain properties defined in Chapter 3. Overstressing for a short period of time to $0.80f_s'$ may be permitted provided the stress, after seating of anchorage occurs, does not exceed $0.70f_s'$.

207.1.2—*Stress at design loads*

Effective steel stress after losses described in Section 208 should not exceed:

$$0.60f_s' \text{ or } 0.80f_{sy}$$

whichever is smaller.

207.2—Non-prestressed reinforcement

Non-prestressed reinforcement provided to resist tension in conformance with requirements of Section 207.3.1.b.2 may be assumed stressed to 20,000 psi.

207.3—Concrete

207.3.1—*Temporary stresses*

Concrete stress in psi before losses due to creep and shrinkage should not exceed the following:

a. Compression

For pretensioned members	$0.60f'_{ci}$
For post-tensioned members	$0.55f'_{ci}$

b. Tension

1. For members without non-prestressed reinforcement:

Single element	$3\sqrt{f'_{ci}}$
Segmental element	zero

2. For members with non-prestressed reinforcement provided to resist the tensile force in the concrete, computed on the basis of an uncracked section:

Single element	$6\sqrt{f'_{ci}}$
Segmental element	$3\sqrt{f'_{ci}}$

207.3.2—*Stresses at design loads*

After full prestress losses, stresses in psi should not exceed the following:

a. Compression
   1. Single element
      a. Bridge members ..... $0.40f_c'$
      b. Building members ..... $0.45f_c'$
   2. Segmental elements
      a. Bridge members ..... $0.40f_c'$
      b. Building members ..... $0.45f_c'$

b. Flexural tension in the precompressed tensile zone
   1. Single element
      a. Bridge members ..... zero
      b. Pretensioned building elements not exposed to weather or corrosive atmosphere ..... $6\sqrt{f_c'}$
      c. Post-tensioned bonded elements not exposed to weather or corrosive atmosphere ..... $3\sqrt{f_c'}$
   2. Segmental elements
      a. Bridge members ..... zero
      b. Building members ..... zero

Allowable flexural tension of $6\sqrt{f_c'}$ in Section 207.3.2.b.1.b may be exceeded provided it is shown by tests that the structure will behave properly under service conditions and meet any necessary requirement for cracking load or temporary overload.

207.3.3—*Stress at cracking load*

Flexural tensile strength (modulus of rupture) should preferably be determined by test. When test data are not available the ultimate flexural tensile stress in psi may be assumed as:

$$f_t' = 7.5\sqrt{f_c'}$$

For lightweight concrete, f_t' should be determined by tests.

207.3.4—*Anchorage bearing stresses*

The maximum allowable stress at post-tensioning anchorage in end blocks adequately reinforced in conformance with Section 214.4 may be assumed as:

$$f_{cp} = 0.6 f'_{ci} \sqrt[3]{A_c/A_b}$$

where A_b = bearing area of the anchor plate.
A_c = maximum area of portion of the member that is geometrically similar to and concentric with the area of bearing plate.

The allowable value of f_{cp} should not exceed f'_{ci}.

208—LOSS OF PRESTRESS

208.1—Introduction

Initial prestress is that stress in steel which exists immediately after seating of anchorage. Stress diminishes with time and finally reaches a stable condition of effective prestress assumed to be permanent.

208.2—Sources of prestress loss

208.2.1—*Friction loss in post-tensioned steel*

If post-tensioned steel is draped, or irregularities exist in alignment of ducts, steel stress will be less within the member than at the jack because of friction between prestressing steel and duct. Magnitude of this friction should be estimated for design and verified during stressing operation.

Friction loss may be estimated from an analysis of forces exerted by prestressing steel on duct. One method for determination of friction loss at any point is given below.

$$T_o = T_x \, e^{(KL + \mu\alpha)}$$

where T_o = steel stress at jacking end

T_x = steel stress at point x

e = base of Naperian logarithms

K = friction wobble coefficient per ft of prestressing steel

L = length of prestressing steel element from jacking end to point x in ft

μ = friction curvature coefficient

α = total angular change of prestressing steel element in radians from jack to point x

For small values of KL and $\mu\alpha$ the following formula may be used:

$$T_o = T_x (1 + KL + \mu\alpha)$$

The following values of K and μ are typical and may be used as a guide. They may vary appreciably with duct material and method of construction. Values of K and μ used in design should be indicated on the plans for guidance in selection of materials and methods that will produce results approaching the assumed values.

Type of steel	Type of duct or sheath	Usual range of observed values		Suggested design values	
		K	μ	K	μ
Wire cables	Bright metal sheathing	0.0005-0.0030	0.15-0.35	0.0020	0.30
	Galvanized metal sheathing			0.0015	0.25
	Greased or asphalt-coated and wrapped	0.0030	0.25-0.35	0.0020	0.30
High strength bars	Bright metal sheathing	0.0001-0.0005	0.08-0.30	0.0003	0.20
	Galvanized metal sheathing			0.0002	0.15
Galvanized strand	Bright metal sheathing	0.0005-0.0020	0.15-0.30	0.0015	0.25
	Galvanized metal sheathing			0.0010	0.20

Workmanship in placing, supporting, tying, and fabricating prestressing elements and ducts influences the magnitude of wobble factor, K. The larger the sheath or duct in relation to the size of prestressing steel element, the smaller K will be. With normal placing tolerances wobble effect may be neglected if sheath is 1 in. greater in diameter than prestressing steel element.

Effect of overestimating friction loss should be considered since excessive prestress may cause undesirable permanent stress conditions. Underestimating friction loss may result in an error in computing cracking load and deflection.

208.2.2—*Elastic shortening of concrete*

Loss of prestress caused by elastic shortening of the concrete occurs in prestressed concrete members. This loss equals $n(\Delta f_c)$. For pretensioned concrete, Δf_c is the concrete stress at the center of gravity of the prestressing steel for which the losses are being computed. For post-tensioned concrete where the steel elements may not be tensioned simultaneously, Δf_c is the average concrete stress along one prestressing element from end to end of the beam caused by subsequent post-tensioning of adjacent elements.

208.2.3—*Shrinkage of concrete*

Shrinkage depends on many variables. Unit shrinkage strain may vary from near 0 to 0.0005. A value between 0.0002 and 0.0003 is commonly used for calculation of prestress loss. Shrinkage loss may be greater in pretensioned members where the prestress is transferred to the concrete at an earlier age than is usual for post-tensioned members. Shrinkage of lightweight concrete may be greater than the values obtained with the above factors.

208.2.4—*Creep of concrete*

Creep is the time-dependent strain of concrete caused by stress. For pretensioned and post-tensioned bonded members, concrete stress is taken at center of gravity of prestressing steel under effect of prestress and permanent loads (normal conditions of unloaded structure).

In post-tensioned unbonded members, stress is the average concrete stress along the profile of center of gravity of prestressing steel under the effect of prestress and permanent loads. Additional strain due to creep may be assumed to vary from 100 percent of elastic strain for concrete in very humid atmosphere to 300 percent of elastic strain in very dry atmosphere.

Creep of some lightweight concretes may be greater than indicated above.

208.2.5—*Relaxation of steel stress*

Loss of stress due to relaxation of prestressing steel should be provided for in design in accordance with test data furnished by the steel manufacturer. Loss due to relaxation depends primarily on properties of the steel and initial prestress. This loss is generally assumed in the range of 2 to 8 percent of initial steel stress.

208.3—Alternate procedures for estimating prestress losses

Two methods are suggested for estimating prestress losses. Method 1 should be used when individual losses may be predicted with reasonable accuracy. Method 2 applies when specific loss data are lacking.

The ultimate strength is not significantly affected by the magnitude of steel stress loss. An error in choosing the loss is reflected in the cracking load and amount of camber.

208.3.1—*Method 1*

The total stress loss in prestressing steel:

$$\Delta f_s = (u_s + u_s + u_d)\, E_s + \delta_1 f_{si} + \delta_2 f_{si}$$

208.3.2—*Method 2*

Loss in steel stress not including friction loss may be assumed as follows:

Pretensioning	35,000 psi
Post-tensioning	25,000 psi

For camber calculations these values may be excessive.

208.4—Lightweight concrete

Losses due to concrete shrinkage, elastic shortening, and creep should be based on results of tests made with the lightweight aggregate to be used.

209—FLEXURE

209.1—Stresses due to dead, live, and impact loads

Prestressed concrete members may be assumed to function as uncracked members subjected to combined axial and bending forces provided stresses do not exceed those given in Section 207.

In calculations of section properties prior to grouting, areas of the open ducts should be deducted unless relatively small. The transformed area of bonded reinforcement may be included in pretensioned members and post-tensioned members after grouting.

For calculation of stress due to prestress in T-beams no definite recommendations are made at this time, but attention should be given to the possibility that the entire available flange width may be included in calculation of section properties.

209.2—Ultimate flexural strength

209.2.1—*General method*

(a) *Rectangular sections*—For rectangular sections or flanged sections in which the neutral axis lies within the flange, ultimate flexural strength may be expressed as:

$$M_u = A_s f_{su}\, d \left(1 - \frac{k_2}{k_1 k_3} \frac{p f_{su}}{f_c'}\right) \qquad \text{(a)}$$

where f_{su} = average stress in prestressing reinforcement at ultimate load
d = depth to centroid of force
k_2 = ratio of distance between extreme compressive fiber and center of compression to the depth to neutral axis
k_1k_3 = ratio of average compressive concrete stress to the cylinder strength, f_c'

The results of numerous tests have shown that the factor k_2/k_1k_3 may be taken equal to 0.6 for members and materials considered in this report. Determination of the value of f_{su} requires knowledge of the stress-strain characteristics of the prestressing steel, effective prestress and crushing strain of the concrete. Assumptions must be made regarding the relation between steel and concrete strains. These assumptions will be different for bonded and unbonded construction.

The ultimate moment may be computed from Eq. (a) whenever sufficient information is available for the determination of f_{su}. The approximate method of Section 209.2.2 may be used if the required conditions are satisfied.

(b) *Flanged sections*—If a flange thickness is less than $1.4dpf_{su}/f_c'$, the neutral axis will usually fall outside the flange and the following approximate expression for ultimate moment should be used:

$$M_u = A_{sr}f_{su}\,d\left(1 - 0.6\frac{A_{sr}f_{su}}{b'df_c'}\right) + 0.85f_c'\,(b - b')t\,(d - 0.5t) \quad \text{.....(b)}$$

where $A_{sr} = A_s - A_{sf}$ = the steel area required to develop the ultimate compressive strength of the web of a flanged section

$A_{sf} = 0.85f_c'\,(b - b')t/f_{su}$ = steel area required to develop the ultimate compressive strength of the overhanging portions of the flange.

t = average thickness of flange

The expressions for f_{su} given in Section 209.2.2 may be used if the required conditions are satisfied.

209.2.2—*Approximate method*

The following approximate expressions for f_{su} may be used in Eq. (a) and (b) of Section 209.2.1 provided the following conditions are satisfied:

1. The stress-strain properties of the prestressing steel are reasonably similar to those described in Section 304.
2. The effective prestress after losses is not less than $0.5f_s'$.

(a) *Bonded members*

$$f_{su} = f_s'\left(1 - 0.5\frac{pf_s'}{f_c'}\right)$$

(b) *Unbonded members*—Ultimate flexural strength in unbonded members generally occurs at lower values of steel stress than in bonded members. Wide variations between stress levels reported by different investigators reflect the fact that several factors influence the stress developed by unbonded steel at ultimate moment. These factors include: magnitude of effective prestress, profile of the prestressing steel, shape of the bending moment diagram, length/depth ratio of the member, magnitude of the friction coefficient between the prestressing steel and duct, and amount of bonded non-prestressed supplementary steel.

Unless the proper value of f_{su} is known from tests of members closely approximating proposed construction with respect to the several factors listed in the preceding paragraph, it is recommended that:

$$f_{su} = f_{se} + 15{,}000$$

209.2.3—*Maximum steel percentage*

To avoid approaching the condition of over-reinforced beams for which the ultimate flexural strength becomes dependent on the concrete strength, the ratio of prestressing steel preferably should be such that pf_{su}/f_c' for rectangular sections, and $A_{sr}f_{su}/b'df_c'$ for flanged sections are not more than 0.30.

If a steel ratio in excess of this amount is used, the ultimate flexural moment shall be taken as not greater than the following values when either the general or approximate method of calculation is used.

(a) *Rectangular sections*

$$M_u = 0.25\, f_c'bd^2$$

(b) *Flanged sections*—If the flange thickness is less than 1.4 dpf_{su}/f_c' the neutral axis will usually fall outside the flange and the following formula is recommended.

$$M_u = 0.25bd^2f_c' + 0.85f_c'\,(b - b')t(d - 0.5t)$$

209.2.4—*Non-prestressed reinforcement in conjunction with prestressing steel*

209.2.4.1—*Conventional reinforcement*—Non-prestressed conventional reinforcement may be considered to contribute to the tensile force in the beam at ultimate moment an amount equal to its area times its yield point provided that

$$\frac{pf_{su}}{f_c'} + \frac{p'f_y'}{f_c'} \text{ does not exceed } 0.3$$

where f_y' = yield point of conventional reinforcement
p' = ratio of conventional reinforcement

209.2.4.2—*High tensile strength reinforcement*—If untensioned prestressing steel or other high tensile strength reinforcement is used in conjunction with prestressed reinforcement, the ultimate moment should be calculated by means of the general method of Section 209.2.1.

210—SHEAR

210.1—General

210.1.1—*Ultimate strength*

It is essential that shear failure should not occur before ultimate flexural strength required in Section 209.2 is developed. If this condition is satisfied, it is unnecessary to investigate shear or principal tensile stresses at design loads.

210.1.2—*Inclined cracking*

Formation of inclined cracks precedes failure in shear. They are caused by inclined principal tensile stresses that are the resultant of shearing stresses and normal bending stresses. Compressive prestress reduces the principal tensile stress thereby increasing the load necessary to cause inclined cracks. The use of thin webs will increase inclined stresses.

210.1.3—*Conditions for shear failure*

The resistance to formation of inclined cracks is greater with larger prestress and increasing web thickness. The significance of inclined cracks is less with low ultimate flexural strength caused by low ratio of reinforcement. Their significance is also less with low shear/moment ratios. If inclined cracks occur in an unreinforced web, sudden failure by shear is almost certain. If the web is adequately reinforced, ultimate flexural strength can be developed.

210.2—Web reinforcement

210.2.1—*Critical percentage of tensile steel*

Experimental data, although limited, indicate that inclined tension cracks will not form and web reinforcement will not be required if the following condition is satisfied:

$$\frac{pf_s'}{f_c'} \geq 0.3\frac{f_{se}}{f_s'}\frac{b'}{b}$$

where b' = thickness of web; b = width of flange corresponding to that used in computing p

This expression may be conservative for members having span/depth ratios greater than about 15 or for uniformly loaded members. In such cases, web reinforcement may not be required even though the percentage index, pf_s'/f_c', exceeds that given in the above expression. The omission of web reinforcement in such members may be allowed when justified by tests.

210.2.2—*Design of web reinforcement*

The amount of web reinforcement necessary to develop required ultimate flexural capacity is a function of the difference between inclined cracking load and ultimate load in flexure. This difference varies rather widely as a function of prestress force, web thickness, amount of tensile reinforcement, and shear/moment ratio but is usually smaller for prestressed concrete than for conventional reinforced concrete. Current design procedures for web reinforcement in reinforced concrete are conservative for prestressed concrete.

Available test data indicate that the following expression for area of web reinforcement, with its factor of ½, will give reasonably conservative results

for prestressed members of usual dimensions and properties. Since the formula does not involve the prestress force it may not be conservative for very low prestress or where only a portion of the reinforcement is stressed. For such cases it may be necessary to increase the factor of ½ as the member approaches the condition of conventionally reinforced concrete.

$$A_v = \tfrac{1}{2}\frac{(V_u - V_c)s}{f_v' jd}$$

where A_v = area of web reinforcement at spacing s, placed perpendicular to the axis of the member
V_u = shear due to specified ultimate load and effect of prestressing
$V_c = 0.06 f_c' b' jd$ but not more than $180 b' jd$
s = longitudinal spacing of web reinforcement
f_v' = yield strength of web reinforcement

210.2.3—*Minimum quantity of web reinforcement*

Because of the nature and limited knowledge of shear failures, it is suggested that some web reinforcement be provided even though the criterion of Section 210.2.1 is satisfied.

Where the web reinforcement is designed by Section 210.2.2, the minimum amount of web reinforcement should be $A_v = 0.0025 b's$. This requirement may be excessive for members with unusually thick webs and the amount of web reinforcement may be reduced if tests demonstrate that the member can develop its required flexural capacity.

Heavily loaded members with thin webs and relatively small span/depth ratios, such as highway bridge girders and crane girders should have web reinforcement (see Section 206.6).

210.2.4—*Spacing of web reinforcement*

The spacing of web reinforcement should not exceed three-quarters the depth of the member. In members with relatively thin webs, spacing should preferably not exceed the clear height of the web.

210.2.5—*Critical sections for shear*

Because formation of inclined cracks reduces flexural capacity the critical sections for shear will usually not be near the ends of the span where the shear is a maximum but at some point away from the ends in a region of high moment.

For the design of web reinforcement in simply supported members carrying moving loads, it is recommended that shear be investigated only within the middle half of the span length. The web reinforcement required at the quarter-points should then be used throughout the outer quarters of the span.

For simply supported members carrying only uniformly distributed load, the maximum web reinforcement may be taken as that required at a distance from the support equal to the depth of member. This amount of web reinforcement should be provided from this point to the end of member. In the middle third of the span length, the amount of web reinforcement provided should not be less than that required at third-points of the span.

211—BOND AND ANCHORAGE

211.1—Pretensioning

211.1.1—*Prestress transfer bond*

Bond between the pretensioned steel and concrete is necessary to establish a prestress in the concrete. The transfer of force from the steel to the concrete takes place in a finite length in the end region of a member and the function of the resulting bond, termed "prestress transfer bond," is anchorage of prestressing steel. Prestressing force varies from near zero at the end to a maximum value some distance from the end.

Transfer length will generally be of minor significance in long members, but it should be considered for short members or those in which the loading conditions may cause cracking in or near the region of prestress transfer.

211.1.2—*Flexural bond*

Flexural bond is the bond stress developed as a consequence of flexure. Bond stress at design loads in uncracked members is usually not critical since the increase in steel stress resulting from flexure is usually not significant. If cracking is anticipated under design loads, bond stress should be given special consideration.

211.1.3—*Significance of bond stress at ultimate load*

Bond failure should not occur prior to the development of the required ultimate flexural capacity.

For span lengths usually associated with prestressed concrete, bond failure is not a significant design factor. Bond adequacy in extremely short members should be investigated by test.

The factors affecting bond are concrete strength, perimeter shape, area and surface condition of prestressing steel, stress in the steel at ultimate strength, length of transfer zone, and superimposed load pattern.

212—COMPOSITE CONSTRUCTION

212.1—Introduction

Prestressed concrete structures of composite construction are comprised of prestressed concrete elements and plain or conventionally reinforced concrete elements interconnected in such a manner that the two components function as an integral unit. The prestressed elements may be pretensioned or post-tensioned and may be precast or cast in place. The plain or reinforced concrete elements are usually cast in place.

212.2—Interaction

212.2.1—*Shear connection*

To insure integral action of a composite structure at all loads, a connection should be provided between the component elements of the structure capable of performing two functions:

(1) To transfer shear without slip along the contact surfaces, and
(2) To prevent separation of the elements in a direction perpendicular to the contact surfaces.

212.2.2—*Transfer of shear*

Slip may be prevented and shear transferred along the contact surfaces either by bond or by shear keys. It should be assumed that the entire shear is transferred either by bond or by shear keys.

212.2.3—*Anchorage against separation*

Mechanical anchorage in the form of vertical ties should be provided to prevent separation of the component elements in the direction perpendicular to the contact surfaces. Web reinforcement or steel dowels adequately embedded on each side of the contact surface will provide satisfactory mechanical anchorage.

212.3—Design of shear connection

212.3.1—*Loading stage*

The shear connection should be designed for ultimate load.

212.3.2—*Magnitude and transfer of ultimate shear*

The shear at any point along the contact surface may be computed by the usual method as $v = (V_u Q)/I$. If the bond capacity is less than the computed shear, full width shear keys should be provided throughout the length of the member. Keys should be proportioned according to concrete strength of each component of the composite member.*

212.3.3—*Capacity of bond*

The following values are suggested for ultimate bond resistance of the contact surfaces.

When minimum steel tie requirements of Section 212.3.4 are followed	75 psi
When minimum steel tie requirements of Section 212.3.4 are followed and the contact surface on the precast element is artificially roughened	150 psi
When additional steel ties in excess of the requirements of Section 212.3.4 are used and the contact surface of the precast element is artificially roughened	225 psi

212.3.4—*Vertical ties*

In the absence of experimental information on the capacity of vertical ties it is recommended that all web reinforcement be extended into the cast-in-place concrete.

Spacing of vertical ties should not exceed four times the minimum thickness of the composite elements, or 24 in. whichever is less. The total area of vertical ties should not be less than that provided by two #3 bars spaced at 12 in.

*Lack of experimental data makes the committee hesitate to recommend a shear stress at the root of a key. Indications are that for keys on bridge girders in current use shear stress at the root of a key as high as $0.3f_c'$ would sometimes be required to transmit ultimate shear force.

For light pretensioned members such as those used for building floors not subjected to repetitive loads the above minimum requirements may be too severe. The committee is not prepared to recommend an amount or spacing of steel for this type member.

212.4—Design of composite structures

212.4.1—*Design of composite section.*

Physical properties of the composite section should be computed on the assumption of complete interaction between component elements. For structures composed of concretes of different qualities, the area of one of the component elements should be transformed in accordance with the ratio of the two moduli of elasticity.

212.4.2—*Beam and slab construction*

If the structure is composed of beams with a cast-in-place slab placed on top of the beams, effective slab width should be computed in the same manner as for integral T-beams.

212.4.3—*Allowable stress with different concrete strengths*

In structures composed of elements with different concrete strengths, the allowable stresses should be governed by strength of the portion under consideration.

212.4.4—*Superposition of stress*

Stresses may be superposed in design calculations that involve elastic stresses. Superposition of stresses should not be used in computing ultimate strength since inelastic action of the material is involved.

212.4.5—*Stress after structure becomes integral*

The properties of the composite cross section should be used in computing stresses due to loads applied after the structure becomes integral.

212.4.6—*Shrinkage stresses*

In structures with a cast-in-place slab supported by precast beams, the differential shrinkage tends to cause tensile stresses in the slab and in the bottom of precast beams. Stresses due to differential shrinkage are important only insofar as they affect cracking load. When cracking load is significant, such stresses should be added to the effects of loads.

212.4.7—*Ultimate strength*

Ultimate strength of a composite section should be computed in the same manner as ultimate strength of an integral member of the same shape.

213—CONTINUITY

213.1—Determination of moments, shears, and thrusts

Moments, shears, and thrusts produced by external loads and prestressing force should be determined by elastic analysis. Effects of axial deformation should be considered. Determination of effects produced by the prestressing

forces should take into account the restraint of attached structural elements and supports.

213.2—Stresses

Allowable stresses are those recommended in Section 207.

213.2.1—*Prestress*

When prestressing is to be applied in more than one stage, the internal stresses should be investigated at each stage.

213.3—Frictional losses

Frictional losses in continuous post-tensioned steel may be more significant than in simply supported members.

213.4—Ultimate strength

The ultimate strength of a continuous member should be evaluated not only at points of maximum moment, but also at intermediate points. In applying ultimate load factors where dead load causes effects opposite to those of live load, consideration should be given to load factor combinations in which dead load factor may equal one. It is recommended that moment redistribution not be considered in design at the present time.

214—END BLOCKS

214.1—Purpose

An enlarged end section, called an end block, may be required to transmit concentrated prestressing forces in a shaped member from the anchorage area to the basic cross section.

End blocks may be required to provide sufficient area for bearing of anchorages in post-tensioned design. They may be needed to transmit vertical and lateral forces to supports and to facilitate end detailing.

214.2—Requirements

In pretensioned members with large concentrated eccentric prestressing elements, end blocks should be used. For lightly pretensioned members, or members of approximately rectangular shape, end blocks may be omitted. However, reinforcement should always be provided in the anchorage zone.

In post-tensioned, shaped members, end blocks should be provided.

214.3—Proportioning

End blocks are usually proportioned by experience. Depending on the degree of concentration and eccentricity of the prestressing force at the end surface, the length of the end block should be from one-half the depth of the member to the full depth. In general, shallow members should have an end block length equal to the depth, and deep beams should have an end block length equal to three-quarters of the depth. Length of an end block can be considered as the distance from beginning of anchorage area to the point where the end block intersects the narrowest width of member.

214.4—Reinforcement

Reinforcing is necessary to resist tensile bursting and spalling forces induced by the concentrated loads of the prestressing steel. A reinforcing grid with both vertical and horizontal steel in the plane of the cross section should be provided directly beneath anchorages to resist spalling forces. Closely spaced reinforcement should be placed both vertically and horizontally throughout the length of the end block to resist tensile forces.

215—FIRE RESISTANCE

215.1—General

The fire resistance of both prestressed concrete and reinforced concrete is subject to the same general limitations. One is the rate of heat transmission through the concrete from the surface exposed to fire to the unexposed surface. The other is the reduction of steel strength at the temperatures induced in the steel during the test. Either limitation may govern.

215.2—Heat transmission

Since the rate of heat transmission through prestressed concrete is similar to that of reinforced concrete of the same composition, the critical dimensions to control temperature rise at the unexposed surface will be the same in prestressed or reinforced concrete members.

215.3—Load-carrying capacity

The ability of the structure to carry required loads during fire test depends largely on thickness of cover over prestressing steel. The following minimum thicknesses of concrete cover on prestressing steel and end anchorages are recommended for various fire ratings:

Hour rating	1 hr	2 hr	3 hr	4 hr
Minimum concrete cover	1½ in.	2½ in.	3 in.	4 in.

Data now available are insufficient to make recommendations for such factors as shape of cross section, type and arrangement of prestressing steel. The cover thicknesses recommended are believed to be conservative.

216—COVER AND SPACING OF PRESTRESSING STEEL

216.1—Cover

The following minimum clear concrete covers are recommended for prestressing steel, ducts, and non-prestressed steel.

	Minimum concrete cover
Concrete surfaces exposed to weather	1½ in.
Concrete surfaces in contact with ground	2 in.
Beams and girders not exposed to weather	
Prestressing steel, and main reinforcing steel	1½ in.
Stirrups and ties	1 in.
Slabs and joists not exposed to weather	¾ in.

216.2—Spacing at ends

216.2.1—*Spacing of pretensioning steel*

Minimum horizontal or vertical clear spacing between pretensioning steel elements at ends of members should be three times the diameter of the steel or 1⅓ times the maximum size of coarse aggregate, whichever is greater.

216.2.2—*Spacing of post-tensioning ducts*

The clear space between conduits at the ends should be a minimum of 1½ in. or 1½ times the maximum size of coarse aggregate, whichever is greater.

216.2.3—*Dimensions of post-tensioning ducts*

When steel is placed inside conduits which are to be filled with cement grout, such conduits should have a minimum inside diameter ¼ in. larger than the diameter of the prestressing steel.

216.3—Draped prestressing steel

When prestressing steel is placed in a curved or deflected position, steel or conduits may be bundled together in the middle third of the span length provided the minimum spacing recommended in Section 216.2.1 and 216.2.2 is maintained for a minimum distance of 3 ft at each end of member. The committee is not prepared to suggest limits for the number of conduits or prestressing steel elements that may be bundled horizontally and vertically. Excessive bundling may lead to insufficient bond capacity in pretensioned members, resulting in bond slip.

CHAPTER 3—MATERIALS

301—INTRODUCTION

The nature and economics of prestressed concrete construction require the use of high strength materials. Ability to sustain high stresses with a minimum of time-dependent change in stress or strain is essential.

These requirements are more severe than those for conventionally reinforced concrete. Highest standards of manufacture and construction should be observed. Prior to adoption of new materials, sufficient test data should be obtained to verify properties assumed in design.

302—CONCRETE

302.1—Scope

Particular attention should be given to properties of individual materials used in prestressed concrete and their effect on compressive strength, modulus of elasticity, drying shrinkage, creep, bond strength, and uniformity of concrete in place.

When new materials and methods are employed, trial mix investigations should include tests for drying shrinkage, creep, and modulus of elasticity.

302.2—Materials

302.2.1—*Portland cement*

Portland cement should conform to one of the following:

Specifications for Portland Cement (ASTM C 150)

Specifications for Air-Entraining Portland Cement (ASTM C 175)

Specifications for Portland Blast Furnace Slag Cement (ASTM C 205)

Specifications for Portland-Pozzolan Cement (ASTM C 340)

302.2.2—*Concrete aggregates*

Concrete aggregates should conform to one of the following:

Specifications for Concrete Aggregates (ASTM C 33)

Specifications for Lightweight Aggregates for Structural Concrete (ASTM C 330)

Mineral composition and soundness of aggregates may have a marked influence on compressive strength, modulus of elasticity, drying shrinkage, and creep.

Concretes made with some lightweight aggregates may exhibit a lower modulus of elasticity, greater creep and drying shrinkage than do concretes of the same strength made with aggregates of normal weight.

The range of properties possible in the same concrete mix with different lightweight aggregates may be large. Therefore, it is recommended that test data should be obtained for compressive strength, modulus of elasticity, drying shrinkage, creep, modulus of rupture, and bond.

302.2.3—*Water*

Water for mixing concrete should be clean and free of injurious quantities of substances harmful to concrete or to prestressing steel. Sea water should not be used for making prestressed concrete.

302.2.4—*Admixtures*

Certain admixtures may be beneficial to fresh or hardened concrete. However, admixtures should not be used until shown by test to have no harmful effect on the steel or concrete.

The use of calcium chloride or an admixture containing calcium chloride is not recommended where it may come in contact with prestressing steel.

302.3—Proportioning, batching and mixing

The proportioning of materials, batching, and mixing of concrete for prestressing should be done in accordance with the *ACI Manual of Concrete Inspection*, the U. S. Bureau of Reclamation *Concrete Manual*, or other comparable regulations including ACI Standards "Recommended Practice for Winter Concreting (ACI 604-56)," "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42);" and "Standard Specifications for Ready-Mix Concrete" (ASTM C 94).

Available materials should be proportioned to produce concrete meeting specification requirements with a minimum water content. Slump of fresh

concrete should be as low as feasible. Cement, sand, and narrow-size ranges of coarse aggregate should be separately batched by weight. Water and some liquid admixtures may be batched by volume with accurate measuring equipment. Close control of all materials and operations is essential.

302.4—Strength

The strength required at given ages should be specified by the designer. Controlled concrete should be used and tested in accordance with Section 304 as modified by Section A602(f) of "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

303—GROUT

303.1—General

When required by job specifications, post-tensioned steel should be grouted to completely fill the void surrounding the prestressing steel with a portland cement grout to insure high flexural bond strength and provide permanent protection for the steel.

303.2—Materials

Grout should be made of either (a) cement and water or (b) cement, fine sand, and water. Mix (a) should be used where the cavity is very small. Either Mix (a) or Mix (b) may be used where the cavity is relatively large. Admixtures should conform to recommendations of Section 303.2.4.

303.2.1—*Portland cement*

Same as Section 302.2.1.

303.2.2—*Sand*

Sand should preferably be a natural quartz sand meeting "Tentative Specification for Aggregate for Masonry Mortar (ASTM C 144)," except for gradation requirements. The sand should pass a No. 30 sieve, about 50 percent should pass a No. 50 sieve, and about 20 percent should pass a No. 100 sieve.

303.2.3—*Water*

Same as Section 302.2.3.

303.2.4—*Admixtures*

Certain admixtures may be beneficial to fresh or hardened grout. However, no admixture should be used until shown by test to have no harmful effect on the steel or grout.

Calcium chloride or an admixture containing calcium chloride is not recommended for use in grouting post-tensioned members.

303.3—Proportioning

Proportions of grouting materials should be based on results of tests made on fresh and hardened grout prior to beginning work. Grout should have the consistency of thick cream or heavy paint. When permitted to stand until setting takes place, grout should neither bleed nor segregate.

304—PRESTRESSING STEEL

304.1—General

High tensile strength steel is required in prestressed concrete to provide necessary internal concrete stresses after losses have occurred. The following four types are in common use:

(a) High tensile strength single wire, applied in the form of assemblies made up of two or more substantially parallel wires. They may be used for either pretensioning or post-tensioning purposes.

(b) Small diameter, high strength strand, shop fabricated, is usually made up of six wires spiraled around a center wire. Small diameter strand is normally, though not exclusively, used for pretensioning purposes.

(c) Large diameter high strength strand is usually shop fabricated with factory attached end fittings for post-tensioned construction. It has 7, 19, 37, or more individual wires.

(d) High strength alloy steel bars are produced by a cold stretching or drawing process. They are currently available in diameters ranging from ½ to 1⅛ in. Alloy steel bars are used principally for post-tensioned construction.

Each type of prestressing steel should be made to distinctly separate specifications, of which the following sections give a general description.*

304.2—High tensile strength single wire

High tensile strength single wire is generally made from high carbon steel hot rolled into rods. It is then heat treated by a process termed "patenting" and cold drawn to produce the required final tensile strength. In its most commonly used form the wire is then stress relieved by a controlled time-temperature treatment that improves elastic properties within the tensile range usually employed in prestressing concrete. It also produces a straighter, more easily handled wire.

High tensile strength wire produced by the oil tempering process is not recommended for use in prestressed concrete.

304.2.1—*Ultimate tensile strength*

High tensile strength wire for prestressed concrete is made to minimum tensile strengths as high as 250,000 psi for a diameter of 0.196 in. Higher tensile strengths are available at smaller diameters and lower tensile strengths at larger diameters.

304.2.2—*Shape of stress-strain curve*

Stress relieved wire for prestressing should display a high yield strength and a reasonable elongation before rupture. Minimum yield strength at 1 percent elongation under test load should be equal to 85 percent of specified ultimate tensile strength. Minimum elongation after rupture should be 4 percent in 10 in. Elongation tests should conform to "Specification for Mechanical Testing of Steel Products" (ASTM A 370-54T).

*The American Society for Testing Materials is currently formulating specifications for prestressing steels.

304.2.3—*Ductility*

Wire for prestressing should be capable of a reasonable amount of cold deformation without failure. It should have a minimum reduction in cross-sectional area of 30 percent at rupture.

304.2.4—*Creep and relaxation*

Data concerning typical creep and stress relaxation properties of the material should be obtained from the manufacturer. Special acceptance tests for individual lots are usually expensive and unnecessary.

Creep tests and short-term relaxation tests do not necessarily represent long-time stress relaxation characteristics.

304.3—Small diameter high strength wire strand

Small diameter high strength strand is normally made of seven wires. A straight center wire is enclosed tightly by six spirally wound outer wires. Because of its small diameter, strand can be given a final stress-relieving treatment similar to that for single wires. This treatment improves elasticity and handling characteristics. Acceptance tests, when required, should be made on the strand rather than single wires.

Physical properties should be based on the total metallic area of all the individual wires. Ultimate tensile strength, shape of stress-strain curve, ductility, creep and relaxation should be the same as described in Section 304.2 (high tensile strength single wire) except as follows:

(a) Minimum elongation at rupture, 3.5 percent in 24 in.

(b) Minimum yield strength at 1 percent elongation under test load equal to 85 percent of specified ultimate tensile strength.

304.4—Large diameter high strength wire strand

Large strand may be made of 7, 19, 37, or more galvanized or uncoated hard-drawn wires, spirally wound. Galvanized strand is most commonly used.

Because large diameter strand cannot be given a final stress-relieving treatment, some of its physical properties differ from those of wire or small strand. Acceptance tests, when required, should be based on properties of the strand rather than individual wires.

304.5—Cold stretched high strength alloy steel bars

These bars are usually made from alloy steel designated AISI 5160 or AISI 9260. After hot rolling, the bars are either heat treated or cold worked. Each bar is then cold stretched to a minimum of 90 percent of the specified ultimate strength.

304.5.1—*Ultimate tensile strength*

High strength alloy steel bars are produced with a minimum tensile strength of 145,000 psi for all diameters.

304.5.2—*Shape of stress-strain curve*

High strength bars for prestressing should have a minimum yield strength at 0.2 percent permanent strain equal to 90 percent of the specified ultimate

tensile strength. Minimum elongation after rupture should be 4 percent in a length of 20 diameters.

304.5.3—*Ductility*

Bars for prestressing should be capable of a reasonable amount of cold deformation without failure. The bar should have a reduction of area of not less than 15 percent at rupture.

304.5.4—*Creep and relaxation*

Data concerning typical creep and stress relaxation properties of the material should be obtained from the manufacturer. Special acceptance tests for individual lots are usually expensive and unnecessary.

Creep tests and short-term relaxation tests do not necessarily represent long-time stress relaxation characteristics.

304.6—Corrosion

Since prestressing steels are susceptible to corrosion, they should be protected during storage, transit, and construction.

The term stress corrosion is applied to the embrittlement of steel that occurs under the combined effects of high stress and some corrosive environments. It may take place without apparent surface impairment.

Normally, steel cast in concrete or properly grouted will not be subject to such corrosion. When post-tensioned steel is not grouted, special precautions should be taken to protect the steel (see Section 404.3.2).

305—ANCHORAGES AND SPLICES

305.1—General

Anchorages for post-tensioning elements now in general use consist of:

Threaded ends and wedge anchors for bars; factory attached end fittings for large diameter strand; button-head, sandwich plate, and conical wedges for parallel lay wire systems; and conical wedges for small diameter strand.

Splices are used primarily for bars and consist of threaded couplings.

305.2—Ultimate strength

Anchorages and splices should be capable of developing the ultimate strength of attached steel elements without excessive deformation.

305.3—Anchorage set

Movement of prestressing steel in anchorage during seating should be stated by the manufacturer and substantiated by test data.

CHAPTER 4—CONSTRUCTION

401—INTRODUCTION

This chapter outlines construction procedures that should result in sound and durable structures.

Prestressed concrete members are composed of high strength concrete and steel. Design stresses are closely controlled, but behavior in service depends

upon the specified concrete being properly placed in forms of the correct dimensions around accurately positioned prestressing steel or ductwork for steel. Construction requires accuracy and care. Deviation from careful workmanship may result in an unsafe structure and should not be condoned.

402—TRANSPORTING, PLACING, AND CURING OF CONCRETE

402.1—General

Quality of the finished concrete members depends on care used in transporting, placing, and curing. Recommended practice is outlined in "Building Code Requirements for Reinforced Concrete (ACI 318-56)," Sections 403-406, and "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)."

402.2—Placing

Low slump, high cement content mixes should be placed in the shortest possible time after mixing is completed to prevent loss of workability.

Concrete should be deposited close to its final position. The method of placement should be such that segregation will not occur.

402.3—Vibration

Internal or external vibration or both are usually necessary to produce dense, well-compacted concrete.

Vibrators should not be used to move concrete horizontally in the form. Overvibration should also be avoided.

When internal vibration is used, vibrator heads should be smaller than the minimum distance between ducts or prestressing steel. Care must be exercised to avoid damage to or misalignment of ducts for post-tensioning steel.

Vibration is not a substitute for workability. Judgment should be used in specifying slump, and approved methods of vibration used to achieve maximum compaction.

402.4—Construction joints

In long cast-in-place members the use of construction joints is recommended (1) to reduce cracking near columns caused by settlement or movement of shoring and falsework, and (2) to allow for shrinkage. In general, joints should be placed near falsework supports.

Construction joints preferably should be perpendicular to prestressing steel. Joints should not be made parallel to prestressing steel unless the provisions of Section 212 (composite construction) are followed.

402.5—Curing

Curing should start soon after finishing. If high temperature curing is used, an initial setting time prior to application of heat should be required. Curing should continue until the required strength for application of the prestress force is reached. Fresh concrete should be protected from rain or the rapid

loss of moisture prior to the curing period. Rapid drying should be prevented until the final design strength is obtained.

When high temperature curing is used, the rate of heating and cooling should be controlled to reduce thermal shock to the concrete.

Where identical precast members are required, curing conditions should be uniform to maintain proper quality control.

402.6—Protection from freezing

During periods of freezing temperatures, ungrouted ducts should be blown clear of water or protected against freezing.

403—FORMS, SHORING, AND FALSEWORK

403.1—General

Quality of concrete members depends on the care used in constructing forms and falsework. Correct practices outlined in "Building Code Requirements for Reinforced Concrete (ACI 318-56)" Sections 501 and 502 are recommended.

403.2—Special requirements

Forms for pretensioned members should be constructed to permit movement of the member without damage during release of the prestressing force.

Forms for post-tensioned members should be constructed to minimize restraint to elastic shortening during prestressing and shrinkage. Deflection of members due to the prestressing force and deformation of falsework should be considered in design. Form supports may be removed when sufficient prestressing has been applied to carry dead load, formwork carried by the member, and anticipated construction loads.

404—PLACEMENT OF PRESTRESSING STEEL AND APPLICATION OF PRESTRESSING FORCE

404.1—General

The location of the center of gravity of the prestressing steel, initial and final prestressing force, and the assumed losses due to creep, shrinkage, elastic shortening, and friction shown on the plans are based on the use of specified materials. Other materials not specified but capable of producing the same results may be used with approval of the engineer.

Unless tolerances for location of the prestressing steel are shown, a variation of $\pm$ 1/8 in. to $\pm$ 1/4 in. depending on size of the member, is suggested as maximum permissible.

404.2—Pretensioning steel

404.2.1—*General*

Steel should be kept clean and dry. Foreign matter, grease, oil, paint, and loose rust should be removed prior to casting concrete. A light coat of

rust is permissible and sometimes preferable provided loose rust has been removed and the surface of the steel is not pitted.

404.2.2—*Measurement of prestressing force*

Pretensioning force should be determined by measuring elongation and checking jack pressure on a calibrated gage. Measurement of elongation will usually give more consistent results. When there is a difference of over 5 percent between the steel stress determined from elongation and from the gage reading, the cause of the discrepancy should be ascertained and corrected.

If several wires or strands are stretched simultaneously, provision must be made to induce the same initial stress in each.

404.2.3—*Transfer of prestressing force*

The force in the prestressing steel should be transferred to the concrete smoothly and gradually. If the force in the wires or strands is transferred individually, a sequence of release should be established by the engineer to avoid subjecting the member to unanticipated stresses. Any variation in this sequence should be submitted to the engineer for approval.

404.2.4—*Protection*

Ends of pretensioning steel exposed to weather or corrosive atmosphere should be protected by a coating of asphaltic material. They should preferably be recessed in the member, coated with asphaltic material and covered with mortar.

404.3—Post-tensioning steel

404.3.1—*General*

The steel should be kept clean and dry. For bonded construction, foreign matter, grease, oil, paint, and loose rust should be removed prior to placing steel in ducts. A light coat of rust is permissible provided loose rust has been removed and the surface of the steel is not pitted.

404.3.2—*Protection*

For general use in unbonded construction, galvanizing may be considered to protect the steel from corrosion when coated with grease or asphalt-impregnated material and enclosed in a sheath. Uncoated galvanized steel may be used when it is accessible for inspection and points of bearing are equipped with special shoes to prevent damage to the galvanizing.

If wrappings and coatings are used on nongalvanized steel, the coating should protect the steel from corrosion during shipment, storage, construction, and after the steel is in place. It should permit movement of steel during stressing with minimum friction. The method of protection should be specified or approved by the engineer.

Anchorage and end fittings should be given protective treatment consistent with that given the prestressing steel. They should preferably be recessed in the member and covered with mortar.

404.3.3—*Placement of steel and enclosures*

Ducts or enclosures for prestressing steel are formed in the concrete using tubing, metallic casings, or other materials. They should be positioned and secured to maintain the prestressing steel within the allowable placement tolerances.

For bonded construction, ducts or duct-forming devices should be free from grease, paint, or other foreign matter. Ducts should be protected against entrance of foreign matter prior to grouting.

Anchorage hardware to be cast in the member should be firmly fastened to forms in the proper location.

404.3.4—*Measurement of the prestressing force*

Values of total elongation, corrected for assumed friction loss and anchorage set, and corresponding jack pressures at various increments of prestress should be supplied by the engineer. When a difference of over 5 percent exists between steel stress determined from the corrected elongation and from corresponding gage reading, stressing operation should cease. If the cause of the discrepancy is neither faulty measurement nor equipment, the engineer should be consulted.

404.3.4.1—*Factors influencing friction*—As prestressing force is applied, friction between prestressing steel and curved enclosure reduces steel stress at points away from the jack. The amount of friction loss is a function of degree of curvature, type and length of prestressing steel, duct material, presence of friction reducing agents, accuracy of placing the duct, and degree of disturbance during concrete placement.

It is the responsibility of the contractor to be aware of these factors. He should use materials specified and insure that the quality of workmanship results in accurate duct positioning with minimum displacement during construction.

404.3.5—*Prestressing in stages*

When the prestressing force is to be applied in more than one stage, excessive concrete stresses should be avoided during intermediate stages. The engineer will designate location and magnitude of the forces to be used for each stage and allowable external loads that may be placed on the member. The contractor should be aware of the significance of overloading the member.

404.3.6—*Anchorage set*

For friction type anchorages the manufacturer or supplier should state the amount of slip normally expected in seating the anchorage device.

404.3.7—*Effect of temperature*

Changes in temperature should have little effect on prestressing reinforcement unless there is a significant temperature differential between concrete and steel.

405—GROUTING

405.1—General

When grouting is specified for post-tensioned members it should completely fill all enclosure voids.

405.2—Mixing

Grout should be mixed in a mechanical mixer. Immediately after mixing, it should be passed through a strainer into pumping equipment which provides for recirculation. Grout should be pumped into the duct as soon as possible after mixing but may be pumped as long as it retains the proper consistency.

405.3—Arrangement of grout pipes

Ducts must be provided with entrance and discharge ports, each of which can be closed. Extension pipes may be used when necessary.

For long members, grout may be introduced at one end until it discharges from an intermediate point. The point of application may then be moved successively forward. Grout may be introduced at an intermediate point if discharge ports are provided at duct ends. The sequence of grouting should be planned to insure complete filling. Devices for bleeding air may be required at high points of the duct profile.

405.4—Test for passage of grout

Free passage of grout from entrance to discharge port must be assured. Tests may be made by pumping water, air, or other fluids through the duct.

405.5—Application of grout

Grout should be applied continuously until it flows steadily from the discharge port indicating removal of trapped air and water. The discharge port should then be closed and grouting pressure maintained for the length of time necessary to insure complete filling of the void. The entrance should then be closed and the pumping nozzle removed.

405.6—Protection against freezing

Adequate precautions must be taken to prevent freezing fresh grout.

406—HANDLING AND ERECTION

Where precast members are specified, methods of handling and/or the sequence of erection should be indicated. When these are not indicated on the plans, the contractor should submit for approval the location of pick-up points, minimum concrete strength when handled, method of transporting, and sequence of erection.

Tentative Recommendations for Prestressed Concrete*

Reported by ACI-ASCE Joint Committee 323

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This report was submitted to letter ballot of the committee which consists of 33 members; 32 members returned their ballots, of whom 32 have voted affirmatively.

†These members constitute executive group of committee.

‡Deceased: served as committee chairman from 1949 until 1955.

Notes and Comments

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SELLING PRESTRESSED CONCRETE FOR BUILDING CONSTRUCTION

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We are entering a new era in prestressed concrete production where hit or miss marketing methods can no longer suffice to keep the larger casting facilities busy. High plant overhead and fixed costs create a need for high volume production just to meet the break-even point. Coupled with this, the recent trend to curb spending for new industrial and commercial facilities and the resultant lower sales puts a severe squeeze on profits.

Many prestressing plants in the United States produce both heavy prestressed units for bridge construction and light members for building projects. Since bridge work cannot always be depended upon to keep the casting operations busy it is often necessary to have the building units carry the major portion of the rigid expense.

Prestressed concrete building units have generally been popular with the customer and considering the natural resistance to a new product, the sales have been very good. Much of the material has been sold to customers who have heard of prestressed concrete and are curious to learn if it can be used for their projects. These customers have taken the initiative to purchase the product. Producers who enjoy this situation do not need a sales organization and many have fared well by having a few men wear many hats. It is rather common to see one man handling sales, engineering, production, and general management in the smaller companies. Although this arrangement works well when plant fixed costs are low and business is good, the "walk-in" customer simply cannot be depended upon for the necessary volume in a larger operation.

A formal sales organization with a planned marketing strategy is a fundamental part of any large manufacturing operation. For example, sales goals and the corresponding production forecasts are an integral part of the management planning in the steel and automobile industries. It is, therefore, reasonable to assume that a similar program would pay off in marketing prestressed concrete.

The sales organization has the obvious responsibility of keeping enough work coming in to provide at least a minimum backlog. But in addition to this it is desirable for the sales force to accept the broader responsibility of charting a future course so as to insure enough annual business, stabilize demand, and provide an input forecast that will help in the planning of the entire manufacturing organization. With a definite marketing plan in mind it is much simpler to give proper direction to the day-to-day sales efforts and the sales representatives can budget their time accordingly. It is possible to borrow a couple of terms from the military, it might be desirable to say that these efforts are either strategic or tactical in nature. That is, the sales work is either useful for its long term effect or it is directed toward an immediate sale. Since building, planning and construction is usually a drawn out process it is rarely possible to make an on the spot sale of any importance. Consequently, most sales work in prestressed concrete is useful chiefly for its results over the long pull. With this idea in mind perhaps we

can analyze what techniques are effective in creating a steady demand for prestressed concrete.

How to Create Demand and Sell

Probably the most difficult job of all is that of trying to introduce prestressed concrete into an area where it has not been used before. Naturally, after it has been used on a few jobs in the area, the job is made a trifle easier. Possibly the first step in promoting prestressed concrete in a new area is to let the potential users know your company is in existence and try to determine who is adventurous enough to try it. One job that can be pointed to as an example has a very important effect on future sales. Of course, the material must have an attractive price if it is to be given even so much as a casual hearing. It is true that prestressed concrete has many advantages over other structural materials but price is still the most important consideration to most customers.

The most likely method of introducing the product is to ask the architect and owner for an opportunity to bid an alternate on an existing plan where prestress could show an advantage. This, incidentally, is also a good way of broadening the use of prestressed concrete in an existing market. Often the architect and engineer will be cool to the idea, especially if it will involve major design changes. It is, therefore, sensible to choose jobs that will require little or no extra design work in order to effect the change. The contractor and the owner are much more likely to be interested in the suggestion if there is a definite advantage. Obviously it is most unwise to ignore the wishes of the architect and the engineer in negotiating an alternate proposal. If anyone is going to be hurt by the change, either personally or financially, it is best to avoid pursuing the proposal.

Once a producing company is in operation and is ready to take on work at a steady rate, the big job for the sales people is to contact the architects and engineers in his locality and attempt to interest them in prestressed concrete. Approaching these people for the first time on a cold call is sometimes an uncomfortable meeting. Architects and engineers are generally very busy people with almost all of their attention directed toward the project or projects that they have currently under way. Consequently, the idea of a salesman dropping by to discuss a new product in general terms is often taken as an invasion on their already overtaxed time. It is, therefore, desirable to approach the architect or engineer with the purpose of using prestressed concrete on one of the specific jobs that he is presently planning. Approached in this way, the designer will usually give the salesman more attention. If he is not interested in using the product on the job the salesman has selected he will often divert the conversation to a discussion of prestressed concrete in general or toward another project on which it may be more practical. In any case the interview is usually more desirable and effective. In order to be in touch with what projects are currently in the design stage it is necessary to review the reports published by F. W. Dodge Corporation or a local builder's exchange. If these bulletins are scanned daily it is possible to keep abreast of the majority of construction projects in the sales territory.

In the early stages of introducing prestressed concrete to a new area it is worthwhile to demonstrate the structural capacity of the material before a selected group of architects, engineers, contractors and other interested persons. These demonstrations, if properly carried out, can be very impressive to someone not familiar with prestressed concrete. If the tests are carried out in the

casting yard, the occasion has another advantage of introducing the equipment and techniques used in the manufacture of the products. It has been proven that the investment required in establishing a suitable testing device will pay for itself many times over.

After a few standard products have been chosen and suitable connection details have been developed which are practical in the local area, it is important to prepare printed material that can be distributed to the designers. The importance of having suitable literature that can be used by architects and engineers cannot be overemphasized, yet it is surprising to find the number of producers who have ignored this need. It should be obvious that few professional men will use a material they know little about. They will invariably reach for the manual which illustrates a familiar product. The literature should preferably show photographs of typical attractive buildings that have both a functional and esthetic appeal. It is also necessary for the products manual to have easy to understand loading tables and three dimensional sketches of connection details. It may be desirable to prepare a manual which can be expanded by the use of loose leaf sheets as other products or details are introduced.

What prestressed products should be shown in the manual or what are the best products to market? This is the question that cannot be simply answered because much depends upon the attitudes of the potential users in the local area, the prevailing construction systems and the competitive picture. Naturally the systems that make up a complete floor or roof such as the double tee, channel, or flat slab systems mean higher gross sales for the producer than do systems that employ joists. It is often necessary, however, to have an economical joist to offer if the producer is going to meet the hard competition of steel joists and gypsum decks that are commonly used in low cost roof construction. In any case it is wise not to offer too many different designs of a similar product. The first problem that arises from providing a wide selection of units is that the designer is confused as to which unit to choose. Secondly the producer has a very difficult job of trying to combine several small jobs with varying specifications into his production schedule. The design refinements of many different depths of section and many combinations of prestressing strand does not pay off from a sales and production point of view.

The producer should take advantage of every opportunity to give good will talks and educational programs before professional groups or other groups who are interested in learning about the product. A program that shows slides or movies of prestressed concrete production and construction with proper explanation can be most helpful. The product salesman should let it be known to professional groups that these programs are available.

After prestressed concrete has been used in the locality for a time it will be necessary for the sales staff to learn how to meet the objections. Probably no objection will be more persistent and emphatic than the objection to camber. Camber is probably the greatest single problem of the light prestressed sections used in building construction. If the camber is uniform and not unusually great there is nothing for the producer to be ashamed of or make excuses for. Camber is characteristic of prestressed concrete and a very desirable feature because it insures structural adequacy and is helpful for drainage in roof construction. The main objection to camber, regardless of its various other guises, is the fact that

most people are not used to it. After prestressed concrete becomes an accepted building material in the area, the objections to camber, per se, will gradually disappear. The salesman should, however, advise the designer of this camber feature so that he can at all times keep it in mind. Generally, it is not desirable from an architectural standpoint to have the camber effect show up on the outside of the building.

Needless to say, camber will not be the only objection voiced against precast prestressed concrete. For some reason there seems to be a natural tendency to expect more in the way of appearance from precast concrete products than is generally provided. Steel forms, better vibration equipment and superior form oils have greatly improved the appearance of precast products. In spite of these improvements the uninitiated is still often disappointed when he first comes in contact with precast prestressed concrete. No amount of lip service can really explain away the objections. For this reason it is highly important that the producer be very careful with the appearance of his products. Ragged ends and edges are a big liability even though these imperfections may not be seen in the finished job. Precast prestressed concrete attracts a great deal of attention from curious people in the transporting and erecting stages of the work. Opinions are often formulated about the material by its exterior appearance. Most of the users of prestressed concrete have no other standard with which to measure quality than by its outward appearance.

The same preoccupation with appearance that many users have can sometimes be turned into a valuable asset for the producer. A well constructed building, made with quality prestressed concrete products has a very strong emotional appeal. The light color, smooth surfaces, and long slender lines often connote a feeling of permanence and quality. The salesman would be foolish to overlook this asset in his photographic work and conducted tours to completed jobs. This aspect of precast products is responsible for far more sales than many people realize.

Advertisements too, should suggest what can be done with prestressed concrete to present an architecturally attractive building. Advertising material, that is production centered, too wordy, and lacking in eye appeal will not excite men's imagination or motivate them to consider the material. The producer might also consider the idea of sponsoring a series of ads explaining the advantages of employing architects, engineers, and licensed general contractors in building construction. These messages can benefit the producer as well as the professionals whose work is too often misunderstood and not appreciated.

Evaluation of Performance

No doubt much could be said about what are and what are not effective methods and techniques for selling prestressed concrete. In any case there is no substitute for common sense, a feeling for the local conditions, creative thinking, and plenty of hard work. But how do you know if your efforts are effective? Since selling is such a subjective area, there are few tangible ways to evaluate performance. The natural tendency is to look at the total yearly sales as the standard. But even this does not really tell if the producer got his share of the total work available.

No evaluating or projected sales planning can be done until the sales organization has a basic history of past sales. This history or accounting should record all of the factors that the producer feels is significant in his future work. He may wish to keep a record of each job sold noting the dollar amount, square footage or linear

CONTRACT NO.	CONTRACT AMOUNT	QUANTITY SQUARE OR LINEAL FT.	PRODUCT USED	LOCATION	TYPE OF BUILDING	PRODUCT SPECIFIED	ARCHITECT	ENGINEER	CONTRACTOR	PERSON RESPONSIBLE	EXPOSED	REPEAT ORDER	REMARKS
278	$6,500	7,300 SQ. FT.	DOUBLE TEE	CENTRAL CITY	INDUSTRIAL	STEEL JOIST	NO	YES	YES	OWNER	YES	YES	
279	$25,200	2,100 LIN. FT.	TYPE "C" BEAM	——	——	——	—	—	—	—	—	—	CLEAR RIVER BRIDGE
280	$2,700	2,900 SQ. FT.	FLAT SLAB	BROWNVILLE	COMMERCIAL	FLEXICORE	YES	NO	YES	CONTR.	NO	YES	
281	$11,200	3,200 LIN. FT.	"I" JOISTS	KINGSTON	COMMERCIAL	"I" JOISTS	YES	YES	YES	ENGR.	NO	NO	

TABLE I.

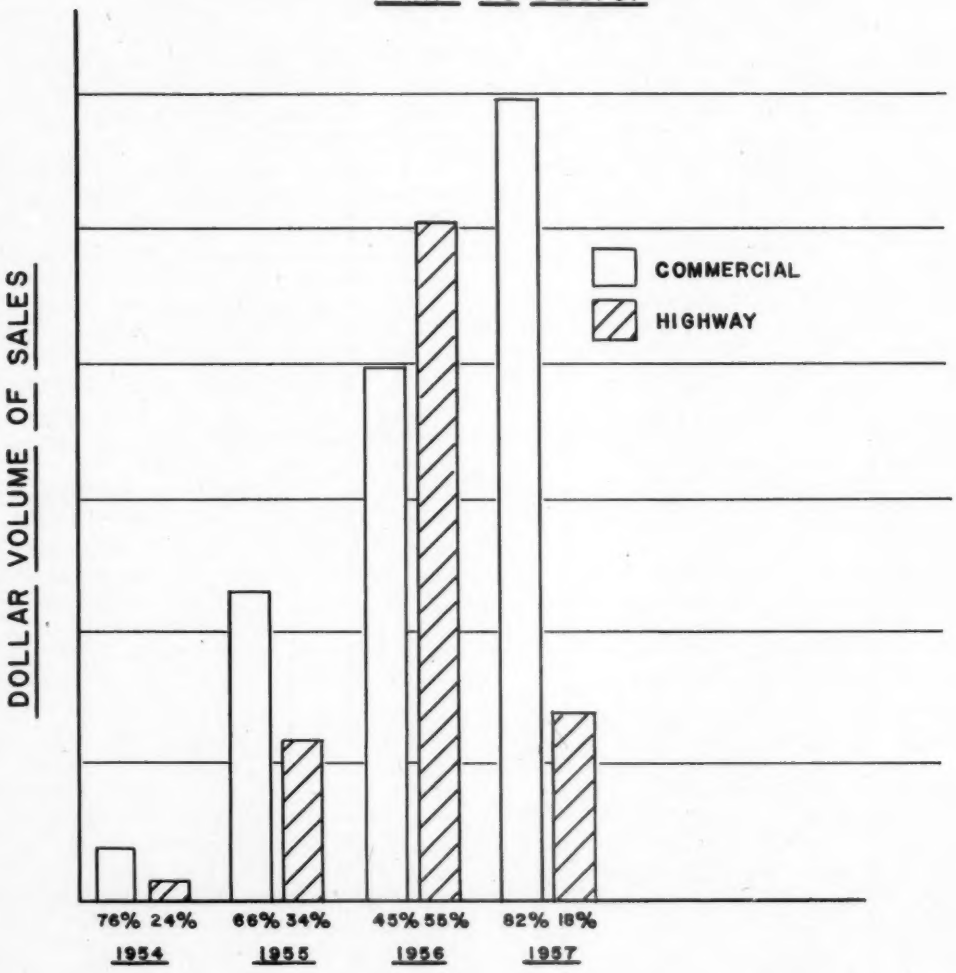
TOTAL YEARLY SALES-COMMERCIAL & HIGHWAY
JAN. 1 TO DEC. 31
DOLLAR VOLUME OF SALES
COMMERCIAL
HIGHWAY
76% 24%
1954
66% 34%
1955
45% 55%
1956
82% 18%
1957

FIG. 1.

footage, location, competitive products, and type of building. He may also want to know if the project employed an architect, engineer, contractor, owner-builder, and who was responsible for putting prestressed concrete on the job. Was the job a repeat order for one of the principals? Was prestress specified on the plans? Did the contractor negotiate the job with the owner or did he bid it competitively? It might also be useful to know if the project used the material exposed. The monthly and yearly accumulation of this information will undoubtedly indicate some significant trends.

Market studies can be conducted in many different ways to bring out a variety of information. An example of a simple sales tabulation sheet is shown in Table 1.

Each contract is analyzed to pick out the key information. For example, contract number 278 was for a roof on a small manufacturing building. The plans called for steel joists but the owner later decided in favor of prestressed concrete double tees. The owner retained a professional engineer to prepare the plans. The order was listed as a repeat because the contractor had used the product before.

Contract 281 was for prestressed "I" type joists used in conjunction with a gypsum deck on a supermarket. An architect prepared the plans and employed an engineer for the structural design. The engineer chose to use prestressed joists even though this was his first use of the product. Neither the architect, engineer, contractor, or owner had used prestressed concrete before.

Records kept on a monthly and yearly basis of the ratio of commercial work to highway work will show the producer which type of work is carrying the load. (Fig. 1.)

By noting the number of jobs that employ architects, engineers, and contractors and by indicating who was responsible for using prestress, it should be possible to determine the group or groups on which to concentrate your sales attention. The number of repeat orders that are received is often significant. It is not uncommon for a producer to have two-thirds of his business coming in from repeat orders. A high proportion of repeat business is an indication of satisfied customers and should point up the need to broaden the base by introducing new people to the products.

A record of the volume of sales kept on a monthly basis may help in the total plant planning. The monthly sales volume may follow a reasonably consistent cycle each year. If such a cycle is definite and clear cut, it can have many overall planning implications. Monthly sales running ahead of normal in a traditionally slow period may indicate a need to expand facilities before the high volume period begins. It may also be possible to find a scheme which would smooth this unbalanced production, either by customer education, increased sales in slow periods, or stock piling.

Conclusion

Prestressed concrete has come of age in building construction and the competitive systems in steel have already felt the impact. Tougher competition probably lies ahead. Producers who hope to extend the use of prestressed concrete in their area must develop and maintain a formal sales organization that can devote conscientious study and aggressive action toward their marketing area. A sales force that simply makes calls and takes orders for material is not enough. Sales planning must also be done if there is to be any contribution to the overall planning of the plant and if an organized course of sales action is to be followed. Marketing progress can only come as a result of careful planning with an eye to the long term. Prestressed concrete simply will not sell itself.

Planning and Design of Installations for Today's Pretensioned Requirements

By CHARLES C. ZOLLMAN

Partner: Charles C. Zollman & Associates, Consulting Engineers

Presented at the 2nd National Prestressed Concrete Short Course Co-Sponsored by the University of Florida and the Prestressed Concrete Institute, January 27-31, 1958

I would like to believe, Mr. Chairman and Gentlemen, that the subject of my presentation is timely and that it has been chosen with great care.

It indeed has become essential for our industry to produce prestressed concrete members of the **highest** quality and at the **lowest** possible cost if our industry is to grow and if it is to develop further — in fact if it is to survive in this keenly competitive market.

I am an old timer, and have been in this prestressing "game" for the better part of twelve years. I have seen this industry develop and grow over the years to what it is today. Therefore, I feel I owe it to you — the producer who intends to expand his present facilities and the potential new producer who considers entering the prestressing field — I owe it to you to let you have my thoughts as to what the technical problems are before you make the move, and what your first move should be, so that the chances for a return of your money invested are the greatest.

Too many installations have been conceived, and I am afraid still are conceived as nothing more than "sidewalks with four posts where strands are to be stretched." It does not occur, until too late, that stretching strands is only a means to the end

and that the real problem is **to produce girders at the lowest possible cost.** Does it really suffice today to provide the producer only with the means of stretching strands? It is not the client who wants stretched strands. The client wants beams and girders, he wants high quality beams and high quality girders and he wants them—CHEAP.

That is what he wanted yesterday. Today he wants more. He wants long span beams and long span girders, to carry more and more load. And these long span beams and long span girders require large numbers of strands with large forces — 1,500,000 lbs. thrust is no longer unusual. To make matters even more difficult for the producer, now he wants draped strands and he challenges the producer to come up with the answer on how to drape them.

The old-fashioned model T bed — the sidewalk and its four posts to stretch strands — does not give the client any longer what he wants, and perhaps never really did! And so it becomes quite evident that an installation for huge forces and draped strands is now a complex affair, if the producer is to produce economically high quality prestressed members.

To be able to produce such members, a plant must be operated efficiently; this is basic.

To operate efficiently, the producer must be furnished with the proper tools and the proper production know-how so that the least amount of labor is required. And, to determine what the proper tools are and who has the proper production know-how, the producer must call for assistance. And, so, gentlemen, the pretensioned installation, which yesterday used to be so often, too often unfortunately, a "do it yourself" proposition, has come of age. At long last, it is the client with his requirements of huge forces and draped strands who compels the producer to think in terms of production and to think in terms of beams and girders to be produced along assembly-line procedures rather than in terms of how to stretch strands. Whereas all the other operations used to be considered as incidental in the planning and design state, it now turns out that it should have been the other way around and that the stressing operation itself is actually a very small part of the entire plant operation — and perhaps the least costly.

Now, how do you achieve an efficient installation? Any engineer with some knowledge of statics can analyze the bed proper; any engineer can determine the size of the uprights, the thickness of the slabs and can compute the dimensions of the abutments. That is no problem! The real problem is to determine the physical components of an installation, their dimensions and sizes in relation to the type of equipment and labor forces which will be required for the handling of the forms, for the setting of the reinforcing steel, for the unreeling and stressing of the strands, for the placing and curing of the concrete, for the handling of the finished product and all of this has to be given thought in relation to the production schedules which must be attained and maintained. That is the real problem. And, when all the various parts and functions which make up the entity of an effective, productive plant have been developed and built by competent construction men, then you must place this tool — because that is what actually a plant is — in the hands of a qualified engineer, if you want to produce at low cost. The finest physical installation will only be as productive as the man who will operate the plant understands prestressing, knows how to make concrete, knows how to use equipment and men to the best advantage.

Summing up, an efficient installation which can produce quality prestressed members at low cost, with the least labor, can only be the result of careful planning, careful design and careful analysis of each component of the plant to be, so that the assembly of all these components in the hands of a competent man will lend itself to effective production.

This planning, gentlemen, let me assure you, is no amateur's job. It takes the joint efforts of men with a thorough knowledge in prestressing, in material handling, in plant operation, in labor relations to bring about an efficient plant. And it takes more than a tour of existing plants to acquire this knowledge, as I could cite you plant after plant, small and large, which has been conceived after such trips and the results are disastrous — strands are stretched all right — but production of quality work at low cost is almost impossible because these installations have not been thought

out and are not provided with the multitude of details which are a prerequisite for efficient production.

The wise potential investor, the intelligent producer is the one who is willing to assume that he doesn't know anything about it — most likely an understatement — but who then surrounds himself with competent men and lets them carry the ball. But then, it takes courage to assume ignorance — most people are generally vain — they know it all — you can't teach them anything. But, that it requires courage is exactly what one of today's largest and most successful producers showed when he had to undertake last summer the manufacture of about 1,300 girders in lengths varying from about 45 ft. to about 90 ft. Some of these girders required up to 79 7/16 in. diameter strands, producing a thrust of approximately 1,600,000 lbs. As many as 35 strands were to be draped. Uplift forces at the hold-down points of the draped strands, which could be located anywhere along the bed depending on the size of the girder, were as high as 35,000 lbs.; downward forces at the bulkheads, again anywhere along the bed, were as high as 120,000 lbs.

That was the task, making these beams, and what were the problems?

The first basic problem was to determine the total bed length required for the casting of all the girders in a given period. This total bed length could only be determined if the time required to complete a production cycle could be known. This in turn is predicated principally on the time needed to carry out all the operations for one cycle and the time required for the concrete to attain its strength.

Once the cycle and the total casting length were determined, a decision had to be made about the number of casting lines into which the total bed length was to be divided; and this again is predicated mainly on the number of each size of girders to be cast, the lengths of forms it would be desirable to purchase, the number of reels it would be well to have, as well as on a number of other appurtenances — so that the projected cycle could be completed within the contemplated time. The choice of equipment enters also in the above decisions. Would cranes, either overhead or mobile, do the job or would such equipment be too costly, too slow or too cumbersome? Or, would lumber carriers, forklifts, travel-lifts do the job? And what about storage areas and connected problem of drainage? What provision should be made for the concrete to attain its strength within a minimum period of time? How should the draping be accomplished? Should strands be pre-draped, i.e., draped before stressing? If so, how is friction, if any, then to be overcome? How will you insure that you have the same stress or about the same stress in the strands at both the near end and the far end of the bed? Or, should strands be draped after their being stressed? How, where and when should the reinforcing cages be made up and installed? What about steam lines, water lines, power lines, communication lines, air lines?

And how should all these operations be coordinated so that effective production could be achieved?

These are only some of the problems and no amateur could solve them properly; no amateur should even attempt to solve them unless he wants to be penny-wise and pound foolish, as some have done and found to their sorrow to be a very expensive procedure. And these problems are the same, perhaps to a smaller degree, for medium and small plants.

It probably now will become apparent to you, as it did to the project manager of this particular job, that perhaps the wisest thing to do would be to plan the design and the operation in its minute details **before** any actual work would commence in the yard. Even though himself a most competent engineer, this particular project engineer understood the need for the services of a prestressing consultant to design and detail the installation, that in view of the limited time available a qualified contractor should build the installation, and that it also would be advisable in view of the magnitude of the job to avail himself of the services of an experienced production consultant to be in charge of the initial operation and to insure that what was being planned, designed and built by others, would lend itself to efficient production.

Prestressing plants are no longer "do it yourself" hit and miss propositions — too much is at stake.

CAMBER CONTROL

By Ray A. McCann

Assistant Chief Engineer, Basalt Rock Company, Inc., Napa, Calif. Registered Civil Engineer — Member PCI, ACI, ASCE.

PRESTRESSED CONCRETE leads to shape refinement and generally a considerable reduction in member size. Like any other flexural member, as the size and section properties are reduced, deflection becomes an increasingly important consideration. By prestressing, it is possible to control deflection. In a roof member for example, some residual upward deflection or camber can be assured. A thorough prestressed concrete design includes investigation of cambers and deflections.

To consider camber, one must first understand creep. For purposes of this discussion only its dependence on stress and time will be considered (for a given mix). If a member is made and maintained with uniform stress for any cross section throughout its length, it will have no camber or camber change. As the member is stressed it will shorten elastically and then gradually creep at a continually diminishing rate. If after a considerable time it is unloaded it will lengthen initially and then continue gradually, however, neither the elastic nor creep changes will be as great as they were under the first loading condition. There will be a "set", due to an increased modulus of elasticity and a decreased creep factor. Usually prestressed members have high compression at the bottom. This compression decreases and often passes into tension at the top. The greater the differential stress the greater the camber. The creep is also greater but not necessarily in direct proportion.

The time-strength variation of concrete is well known. In high strength concretes the modulus of elasticity is not only disproportionate to strength but to time also. It is relatively easy to accelerate strength gain but not as easy to control modulus of elasticity or its increase. Different concretes of the same strength don't necessarily have the same modulus of elasticity.

The primary requirement for uniform control of camber is a laboratory to design mixes and control curing. Once the optimum is determined it is essential to maintain

uniform casting procedures. Herein lies the most important factor in maintaining uniform cambers. Adequate facilities and experienced personnel must be maintained. In addition, accuracy of tensioning and uniformity of mixing, placing, and curing of the concrete must be conformed to. Members should not only be stripped at similar times but stored under similar conditions. Sometimes it proves beneficial to bind members together at an early age so that tendencies for different cambers will work against and counteract each other. The early creep is not only greater but to a large extent determines the final camber condition, since as concrete ages, the creep is less.

Similar members that are stored loaded at an early age will have less camber after removal of load than members that are stored unloaded. To increase final camber, members can be stored with supports inboard of their usual location, provided allowable stresses are not exceeded.

The designer also has some control of camber. The most obvious ways being to vary the amount and location of the prestress force and to prevent bond at the ends of part of the strands. In adjusting the prestress it is important to remember the derivation of deflection formulas: i.e. the statical moment of the area under the moment curve taken about the point considered (e.g. center of a simple span). In other words a given negative moment (or stress differential) at the end of the span has a greater camber effect than the same moment near the center. Members with curved or harped tendons usually have less camber than those with straight tendons.

The designer can also vary the camber by prudent use of mild steel to increase the section properties of the member. One square inch of mild steel being equivalent to eight to fifteen square inches of concrete (depending on the ratio of the moduli of elasticity) it follows that its effect can be quite large. If the steel is in the compression area it may be necessary to use heavy stirrups to prevent buckling.

The designer is obligated to an accurate determination and appraisal of superimposed loads. Usually codes require load capacities which are the highest that can be expected to occur but only a very small portion of this loading may be permanent. To minimize camber changes, members should be designed so that the stresses are very nearly uniform under permanent loads.

To have full camber control, specifications should not be too rigid. Sometimes it is desirable to design a member so that if the full load is applied the extreme fibers will be high in tension. Of course safety is paramount and ultimate load factors must be maintained. For example, a roof slab, designed on the pretense of being conservative, with no tension in the bottom fiber will generally have a greater camber and creep than one allowing tension for the loads which seldom, if ever, will be applied. Sometimes high camber and creep are acceptable. Other times they are not, such as when partitions frame into the prestressed members, or with multi-span conditions when an uneven eave line would result.

The points mentioned here are only qualitative since they depend on the specific conditions of any design. Determination of these conditions necessarily has to be left to the individual judgment of the designer with respect to a particular problem. Once the design is established it is the responsibility of the producer to maintain uniform procedures in order to achieve uniform cambers.

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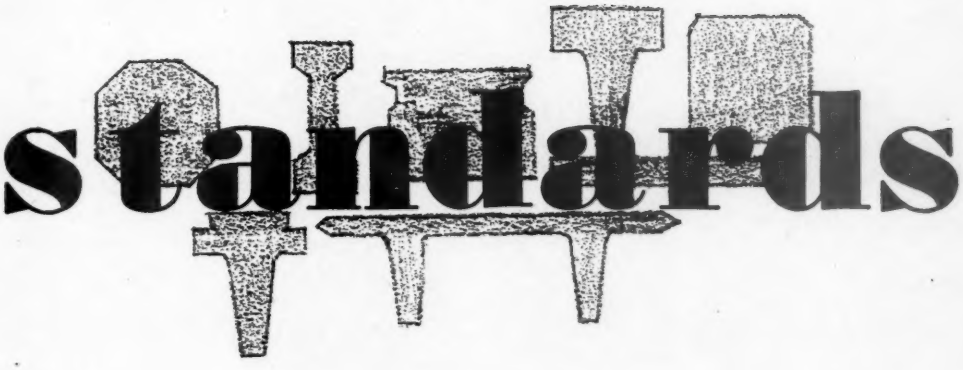

for the prestressed concrete industry

BY H. H. EDWARDS, P.E.
President of Leap Concrete

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Presented at the 2nd National Prestressed Concrete Short Course Co-Sponsored by the University of Florida and the Prestressed Concrete Institute, January 27-31, 1958.

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One year ago, a prediction was made that the prestressing industry will rival the structural steel fabricating industry in size. I would like to change that prediction to say it will surpass the steel fabricating industry and will do it in the not too distant future.

As an example of this rate of growth, consider the amount of prestressed production in Tampa. Two prestressing plants in Tampa will probably produce $6,000,000 worth of products in 1958. The output of these two plants is concentrated on standardized products. An investigation of other high volume operations would show that success in prestressing has been built upon standardization. There is no question as to whether or not the industry will standardize. It has already standardized. Standardization was necessary even to get the industry started. The problem now is what

standard sections will become recognized for universal use throughout the country.

I have here in my hand the steel handbook of "Steel Construction" published by the American Institute of Steel Construction. The engineer is so used to the standards set forth by this book that he is inclined to think that these standards are of long standing in the steel industry. They are not. The American Institute of Steel Construction was founded in 1921. The first manual was based on the Standard Specification for the Design, Fabrication and Erection of Structural Steel for Buildings as adopted by the Institute in 1923. Further revisions were made in 1936, 1941, 1952 and 1956. Previous to this time, each mill published its own manual of standard sections. Each manual presented a different selection of sections. The decision to publish a manual of sections to cover the entire industry was a great boost to the steel industry. It gave rolled steel sections a new standing in the eyes of the design engineer.

Prestressing today is in a similar position to where the steel industry was in 1900. Each firm is still introducing entirely new sections. It is offering its own standards to the engineers in its trade areas. A few groups such as the Leap Associates, and those working with other consultants, have developed their own standards that have gained wide acceptance. For example, 52 prestressed concrete products manufacturers now offer the 14 inch deep by 4 foot wide Double Tee roof slab. A great many firms produced the same cross section of Tee joists and Keystone joists. Bridge sections have made excellent progress toward acceptance of common sections throughout the country.

It is natural and proper that standard sections should develop in both the steel and the prestress industry as they have. A period of development, trial and acceptance is desirable before too great a movement for nationwide standardization is attempted. During this period, the individual firms or associated firms will of necessity have to publish their own standards and manuals. From this group will finally evolve the standards for the entire industry.

Before standards are frozen, we should be well through the inventive and developmental period of the industry. We need to be settled on specifications and criteria for design. Furthermore, time is needed for the assimilation of design techniques by the industry. A better insight into the economic of design and fabrication will undoubtedly force a change in standards.

New prestressed products are continually being introduced. Some of these have a great future. Requirements for fireproofing have not been clearly set forth. Thus we see a period of development ahead which will change our standards. Until this development phase has progressed further, we should encourage new products and new sections and discourage too great a freezing of standards on a national scale.

It would be nice for the prestressed concrete fabricator of today to have a crystal ball to give him a picture of the industry's accepted standards in five and ten years in the future. I am sure no one can make a correct prediction at this time. However, we can give some very good indication which will influence the future standards.

Custom, or firm entrenchment of existing sections, will influence future standards. The Double Tee is today so popular that it is assured a standing with the standards of tomorrow. The most popular section now has a 2 inch thick flange 4 feet wide. It uses a 12 inch stem with a 2½ inch width at its base and a slope of 1 in 12 inches.

Slight changes in one or more of the above dimensions will improve the application for specific conditions. For example, a high fire rating requires a thicker stem, however, this results in the use of more concrete and steel and an increase in the selling price of the product. For extra long spans, a deeper stem is required. Therefore, there are many variations.

In selecting a new standard for production, an individual producer today has several guide posts that can be used effectively. These are as follows:

(1.) Ease of forming. The selection of cross sections should lend itself to ease of forming both from the initial fabrication of the form as well as the labor required in the daily use of the form. The Tee sections have found great popularity in the industry. One of their great assets is that they lend themselves to a very simple form which can be permanently set up. The product may be poured into these forms and removed from the form without disassembling or realigning. This feature of

easy forming applies to the Double Tee, the Tee joist, the channel and the Keystone joist.

(2.) The loading demands will dictate very strongly the shape or depth of the section to be selected. For example, if there is a strong market for heavy loads and long spans in floor construction, then one would wish to consider more seriously the selection of a Double Tee having a deeper stem. However, by making this selection, the producer has then automatically placed his short spans and light loads in a higher price bracket.

(3.) Span requirements should be considered in selecting standard members. For example, if roof span requirements are predominantly in the 40 foot span range and less, then one would do well to consider the 4 foot wide Double Tee having a 14 inch over-all depth of section. However, if the market is strong for the ranges of 40 to 60 feet then a deeper section should certainly be considered.

(4). Fire rating must of necessity be considered before standard building sections are arrived at. Decision on fire rating is very difficult to make at this time. No authoritative standards have as yet been established which will indicate the fire rating to be obtained for any specific amount of concrete coverage over the steel. There is great danger that the individual plant, in making a decision on his standards for steel coverage, will select too high a figure and thereby price himself out of the high volume market. It must always be kept in mind that prestressed concrete, even with low coverage, will obtain a low fire insurance rate, far lower than exposed structural steel construction.

(5). In selecting standards, camber control must certainly be considered. Certain sections are inherently well suited for camber control while others present considerable difficulty. Also, consideration must be given to the sensitivity of the design to bowing or side warpage of the product. Slender joist sections have a tendency to bow and require very accurate form alignment and extremely accurate placing of the prestressing steel.

(6.) Deflection considerations must be considered in selecting standards to be sure that deflections from design load or from overload are not excessive.

(7.) Machine production should certainly be considered. Sufficient development work has now been done on horizontal slip-forming to indicate that it will be used quite extensively by the industry within the next three to five years. Certain sections such as the Keystone joist and Tee joist lend themselves well to production by the horizontal slip-forming. These sections would permit an easy change from the forming system in pans to the slip-forming system if and when it becomes popular.

(8.) Break-even points should be considered for each new product to be produced. Such a chart of cost and selling price plotted against percentage capacity of production will indicate the break-even point to be expected. Such a chart will indicate that the high profits in prestressing are to be made when operating each facility at 100% of productive capacity. Certain sections such as the Keystone joist lend themselves to standard lengths and to a production program for standardizing thus permitting an approach to 100% of capacity production and to the high profit ranges.

(9.) Versatility is one of the requirements for a prestressing plant catering to the building industry. It is necessary to select standards that will cover a wide range of spans and loadings. It is also desirable to offer a joist which may be used with conventional deck materials.

The industry has now accepted several basic cross sections. All of these sections have their particular field of application and would appear to be well established and will gradually be accepted and incorporated into the nationwide design standards sometime in the future. For roof and floor slabs, the industry is now using the Double Tee, the channel and the flat slab. Some use has also been made of the prestressed joist with a poured-in-place decking.

Of all the deck slabs, the Double Tee seems to offer the greatest economies and the lowest price. It may be used in a wide range of spans and loadings although for economy purposes, it competes better on the longer spans. However, it is used quite economically down to as low as 20 feet and may be used in the longer spans up to as high as 70 feet. Generally speaking, the more economical Double Tee will have a thin width of web and will have a depth of stem that is suited for the span and loading. Therefore, the 2½ inch width stem at the bottom with a 12 inch depth

of section has worked out to be a very satisfactory selection for many marketing areas. This is particularly so when the spans are predominantly 40 feet and less.

For floor construction, the Double Tee with a 2 inch poured-in-the-field topping has been very popular. There are now some proposals to produce Double Tees with a 4 inch thick top flange, to place this member on the building and pour a three inch deep edge joint with an expanding grout or an epoxy alloy and then finish grind the floor in place similar to the grinding operation used in making terrazzo floors.

The channel affords a deeper grout key at the edge joint. It is well suited to heavier load construction and to floors having higher concentrated loads. However, the channel section as a rule will cost more than the Double Tee section.

Flat slabs promise to have a very bright future in the prestressing industry. In the use of flat slabs, it is important to use sufficient thickness so that the member does not become too critical on camber control. The 4 inch thick flat slab for use on both floor and roof construction has been quite satisfactory in spans up through 20 feet.

For very heavy floor loads, it is possible to produce a channel or Double Tee section in the conventional Double Tee form having a very thick top flange and designed for flange bearing rather than for stem bearing. These top flanges can be made 4, 6, 8 or 10 inches in thickness and will support very heavy superimposed loads. They also lend themselves well to blast resistant designs.

For joist construction, three basic cross sections have become popular. These are the Tee section, the Keystone section and the I section. The I is basically a very fine structural member. It is used quite extensively for both roof and floor construction and is used in spans from 16 feet on up through 70 and 80 feet in length.

The Tee section for joists appears to be gaining in favor very rapidly. The Tee has several inherent advantages. The first is that it may be made in a very simple form which when once set up and aligned accurately and held in this position may produce thousands of feet of product without ever disturbing the form. The Tee section is ideally suited for prestressing. The Tee may be easily designed so that on ultimate load, the steel will always govern thus giving a deflection warning and eliminating the hazard of brittle failure of the concrete. The Tee also has a very pleasing architectural appearance and is more suited to exposed joint construction than the I section. The Tee may also be made with quite a wide top flange to give better control of lateral stability for handling and erection and to lessen the sensitivtiy to side warpage. The Tees lend themselves to a wide span range starting at 20 feet and going up through 70 to 80 feet.

The Keystone joist may be made in very small sections and is ideally suited for short spans and light loads. The Keystone finds its large application in exposed beam ceiling construction using one of the conventional roof decks which is also left exposed to give a good texture and to furnish insulation and acoustical control. The Keystone will be used very widely for school buildings, stores, offices and also in home construction.

There has been less standardization of beam and girder construction in prestressing than in any other of the building members. It is too soon at this time to tell definitely which type of beam cross section will predominate. At the present time, we have seen good examples on the use of the I beam, the rectangular beam, the inverted Tee beam, the Tee beam and the Ledger beam. It would appear that standardization will come very much later to the beams and girders. These members, of necessity, must often be custom designed to suit the job condition while it is relatively easy to select a short, standard member spanning from beam to beam.

Standardization of prestressed members on a nationwide scale is definitely needed by the prestressing industry. It is felt that this condition can be brought about very easily in another three to five years by selecting the better sections then in common usage by the industry. This means that the industry at that time will need to agree on and publish a design manual which can be used throughout the nation by the architectural and engineering profession. When this has been accomplished, we can truly say that prestressing has arrived at its proper place in the building field. At that time, it will also be clearly recognized that prestressing has outdistanced the structural steel fabricating industry.

a method for prescribing the prestressing force

by W. J. JURKOVICH, Senior Bridge Engineer
Division of Highways, Sacramento, California

WITH prestressed concrete structures becoming more popular the method of showing details and prescribing the prestressing force should be standardized as much as possible. At present some engineers specify initial force and others the final or working force which is defined as the force remaining after losses due to creep and shrinkage of concrete and creep of steel have occurred. If more than one type of prestressing steel is permitted then specifying the initial force is not recommended as a different force is required for the various types of prestressing steel.

Assume for a specific design the required working force is 480,000 lbs. in order to have zero stress in the bottom fibre and 2000 psi maximum compressive stress in the top fibre under full design load. The force of 480,000 lbs. is required to remain permanently. Naturally the initial force must be larger than 480,000 lbs. in order to provide for the losses which result from creep and shrinkage of the concrete and creep of steel. If various types of steel are permitted to be used then specifying a 15% loss is not correct because regardless of type of steel used the loss of steel stress is approximately the same. For example, if alloy bars with an ultimate strength of 145,000 psi are used the working stress would be .6 × 145,000 = 87,000 psi. From the Bureau of Public Roads Criteria for Prestressed Concrete Bridges it is assumed that a loss of 24,000 psi is calculated from the empirical formula, $3000 + 11\ f_{cs} + .04\ f_{s1}$. Note that this formula does not distinguish between the various types of the steel. The area of alloy bars required $= \frac{480}{87} = 5.52$ sq. in. and the initial force would be (87,000 + 24,000) 5.52 = 612,000 lbs. This loss is $\frac{612\text{-}480}{612}$ or 21.6%.

For a wire system with 200,000 psi ultimate strength, the working stress is .6 × 200,000 psi and the area of steel required $= \frac{480}{120} = 4$ sq. in. The initial force would be (120,000 + 24,000) 4 = 576,000 lbs. The loss is $\frac{576\text{-}480}{576}$ or 16.7%.

For a wire system with 250,000 psi ultimate strength the working stress is .6 × 250,000 = 150,000 psi and the area of steel required = $\frac{480}{150}$ = 3.20 sq. in. The initial force would be (150,000 + 24,000) 3.2 = 556,000 lbs. This loss is $\frac{556\text{-}480}{556}$ = 13.7%.

From the above it is obvious that an initial force based on 15% losses cannot be specified if various types of steel are used. Since it is quite simple to specify the the final force and a specific loss of stress in the prestressing steel in pounds per square inch it is recommended that this procedure be adopted. If, however, only one ultimate strength of steel is specified then the percentage loss is not only a correct procedure, but it simplifies the calculations.

It does not appear necessary to substitute in the empirical formulae given in the BPR Criteria to calculate losses for each and every design. It is known that these losses may vary between 20,000 psi to 30,000 psi. It is reasonable to assume a fixed loss of 24,000 psi for all cases. Regardless of what system is used in calculating these losses if they actually come within 3,000 to 4,000 psi of the actual losses the accuracy is sufficient.

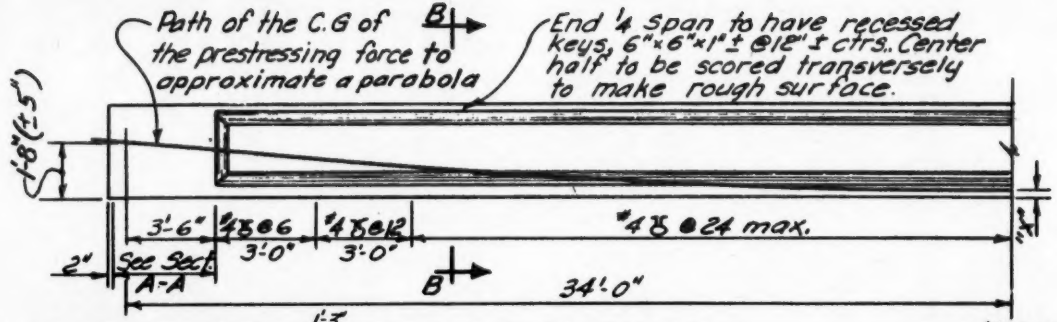
An examination of the BPR formula for losses (3,000 + 11 f_{cs} + .04 f_{s1}) and a review of the discussion on how it was developed (See BPR "Criteria for Prestressed Concrete Bridges," pp 12 & 13) indicates that the losses calculated will be greater than actual in most cases. The first and last terms of the equation appear reasonable. The 11 f_{cs} is based on 2.25 times the elastic deformation. The term "f_{cs}" is defined as the concrete stress at the centroid of the steel. This will give high values for short spans with large live loads and possibly low values for long spans with relatively small live loads. Since the change in stress in the steel is based on 2.25 times the initial deformation it appears more reasonable to calculate the initial deformation using the initial average stress $\frac{Pi}{A}$.

Another argument in favor of using $\frac{Pi}{A}$ instead of f_{cs} is that the concrete stress at the steel varies through the length of the member and an "average" value would not mean the same thing in this case to all engineers.

Actually, if $\frac{Pi}{A}$ were used the losses can be fixed as previously discussed since in most cases $\frac{Pi}{A}$ varies between 1,000 psi to 1,600 psi. For $\frac{Pi}{A}$ = 1,000 psi the theoretical loss would be 28,000± psi. Therefore an average value of 24,000 psi appears to be reasonable for all but a very few cases.

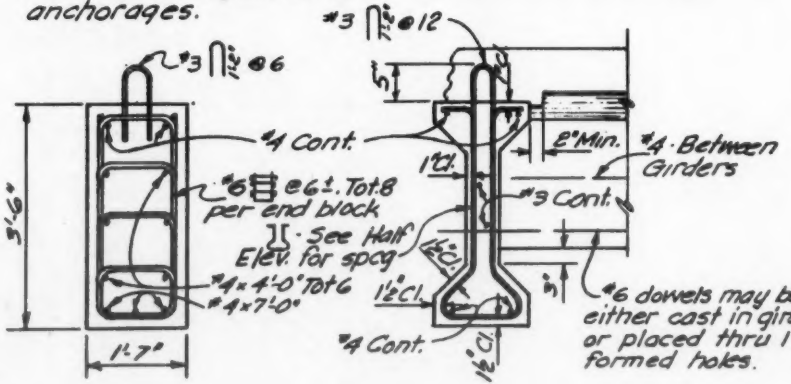
It is recommended in cases where the type of prestressing steel is not limited to one, the center of gravity of the prestressing steel be shown on the plans as indicated in Figure 1. Note that various values of "X" are shown on the plans. The purpose of the procedure is to permit the contractor to vary the eccentricity to fit the system of prestressing he elects to use. The effect of the various eccentricities can be easily evaluated. At the time of stressing the larger the value of "X" the smaller the top fibre tensile stresses or the larger the top fibre compressive stresses at midspan. The bottom fibre compressive stresses at mid span will remain the same.

In order that the contractor can more easily fit in the anchorages in the end block it is recommended that the plans show the preferred location at the supports with a permissible variation on either side. As long as the location of the center of gravity of the prestressing force at the ends is in the kern there is usually no reason for limiting the location of the force to one point.



Note: Place grillage #3 @ 2" ctrs. adjacent to anchorages.

HALF ELEVATION


#3 @ 12
#3 @ 6
#4 Cont.
#6 @ 6 ± Tot 8 per end block
See Half Elev. for spcg
#4 x 4'-0" Tot 6
#4 x 7'-0"
#4 Cont.
2" Min.
#4 Between Girders
#3 Cont.
#6 dowels may be either cast in girder or placed thru 1" ϕ formed holes.
5'-6"
1'-7"

SECTION A-A

SECTION B-B

FIGURE 1

NOTES

Working force (The force remaining after all losses, including those due to creep and shrinkage of concrete and creep of steel have occurred) for the various values of "x":

"x"	"Pf"
3"	476,000 #
4"	495,000 #
5"	515,000 #

The loss in stress in the post-tensioned prestressing steel due to shrinkage, creep and sequence stressing shall be assumed to be 24,000 psi.

In addition to the 24,000 p.s.i. loss noted above, provision shall be made for losses due to friction, anchorage take up, and any other losses peculiar to the system of prestressing used.

Loss in stress for pretensioning system shall be assumed to be 30,000 psi.

Concrete: f'_c = 4,500 psi @ 28 days
f'_c = 3,500 psi @ time of stressing

Prestressing Steel: See Special Provisions.

Prestressed concrete Sea-Wall during construction

Design of Prestressed Concrete Harbor Structures

A. M. Ozell* and W. D. Heasley**

*Professor, Civil Engineering Department, University of Florida, Gainesville, Fla.
**Formerly with Waylan Engineering Co., now with Portland Cement Association, Orlando, Fla.

The use of prestressed concrete elements on harbor structures provides for crack-free, durable and economical structures. An example of such a structure is described below with only the highlights of the design and construction features presented.

The total length of the sea-wall was 2,500 ft. requiring 22 ft. long king-piles for 900 ft. length of the wall and 28 ft. long piles for the remaining portion. A typical cross-section of the wall and the loading diagram used in the design are given in Fig. 1.

The bids were submitted on steel sheet piling (foreign) and the prestressed concrete king-piles and slabs. The prestressed concrete bid submitted by Juno Prestressors, Inc., West Palm Beach, Florida, came in at about 20 percent less than the steel sheet piling.

Fig. 2 shows a typical section and elevation and Fig. 3 gives the cross-section and details of the king-piles and the slabs.

The depth of penetration shown in Fig. 1 was by use of a conventional formula for sheet piling. The final pile design shown in Fig. 2 shows how 2 feet were saved on the slab depth by taking the extra 4 feet on the king-piles into account in the calculations of pressures, which resulted in material savings and greater ease of placing.

The king-piles were driven in place by jetting while the adjustment and spacing of each pile was held true by means of a jig shown in Fig. 4. This particular jig set-up was not completely satisfactory, however, as it did not prevent the piles from spreading out at the bottom. It is believed that a deeper jig would have provided for better control. The equipment used in the placing of the king-piles and slabs consisted of one crane and one pump. The crane operator and two men formed the working crew. An average of six piles and slabs were placed during one working day.

Fig. 5 shows the king-pile with the tapered slots provided for the slabs. The slabs extended only part of the pile length. Fig. 6 shows the slab with tapered sides and half-round furnished for grouting purposes.

SAND IN LOOSE CONDITION - ASSUME $\phi = 30°$

$\gamma_{DRY} = 100$ PCF

$\gamma_{SATURATED} = 125$ PCF

$\gamma_{SUBMERGED} = 62.5$ PCF

$K_A = \tan^2 (45° - \phi/2) = 0.33$

HARBOR BOTTOM INCLINED 1:4 ($\beta = 14°$)

$$K_p = \frac{\sin^2(\alpha-\phi)\cos\sigma}{\sin\alpha \sin(\alpha+\sigma)\left[1-\sqrt{\frac{\sin(\phi+\sigma)\sin(\phi+\beta)}{\sin(\alpha+\sigma)\sin(\alpha+\beta)}}\right]^2}$$

USE $\sigma = 0$, $\alpha = 90°$

$$K = \frac{\sin^2 60°}{\left[1-\sqrt{\frac{\sin 30° \sin 16°}{\sin 30° \sin 76°}}\right]^2} = 3.44$$

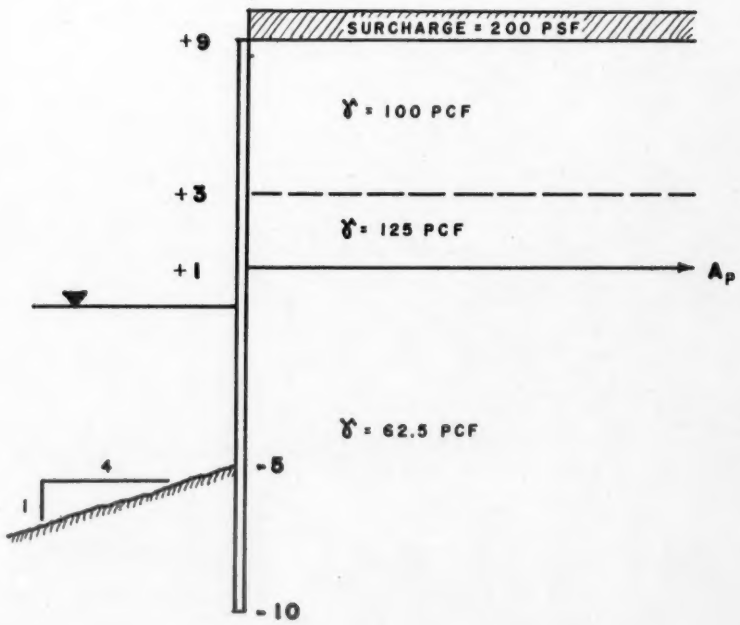


FIG. 1 - TYPICAL CROSS SECTION AND EARTH PRESSURE DIAGRAM

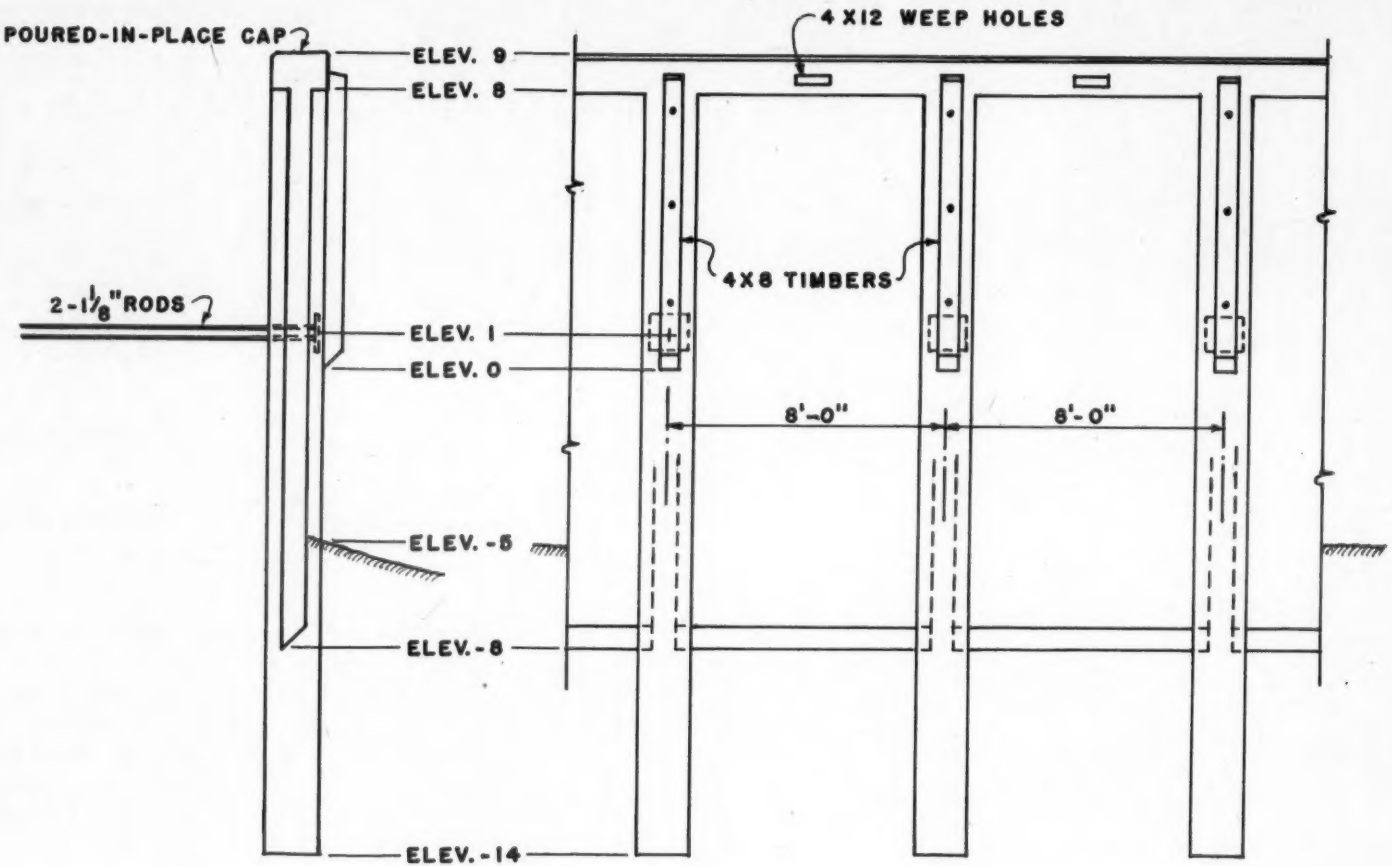
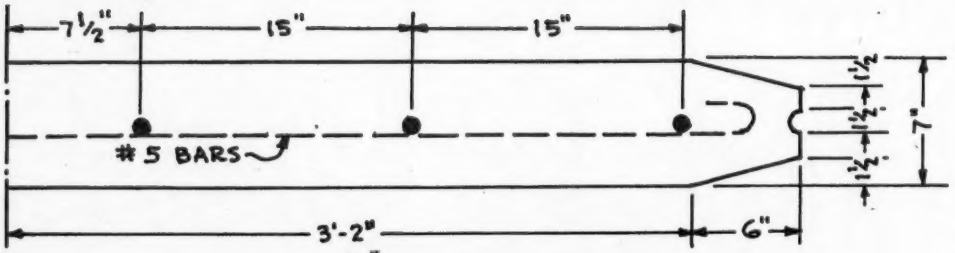
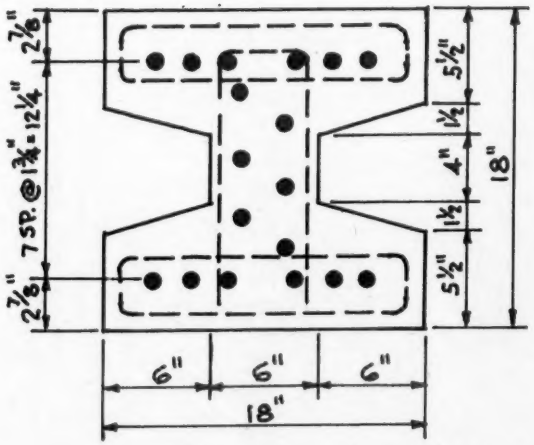


FIG. 2 - TYPICAL SECTION AND ELEVATION



6 - 3/8" STRANDS PER SLAB



18 - 3/8" STRANDS PER PILE

FIG. 3 - CROSS SECTION OF SLABS AND KING PILES

Figure 4—Jig used in the alignment of King-Piles

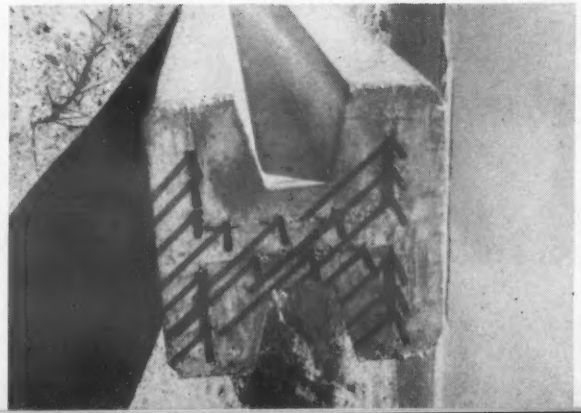


Figure 5—Prestressed concrete King-Piles

Figure 6—Prestressed concrete slabs

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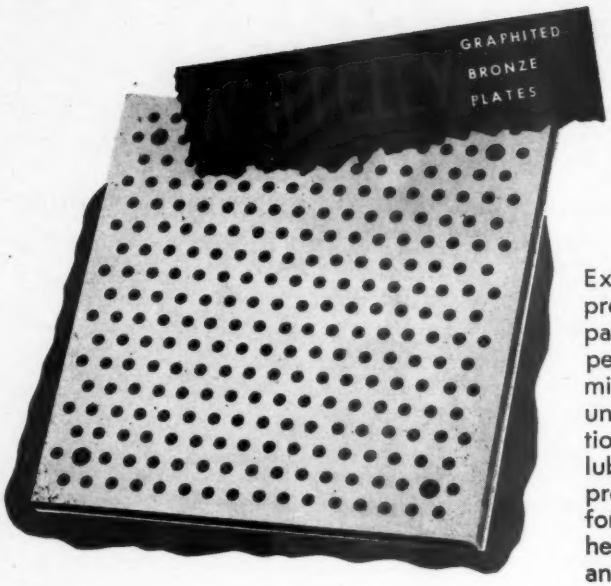

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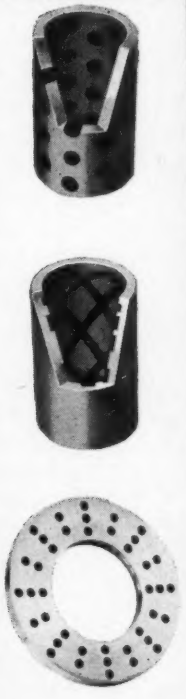

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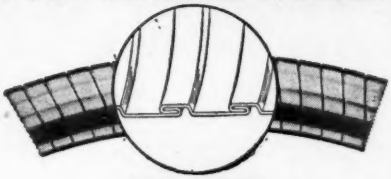

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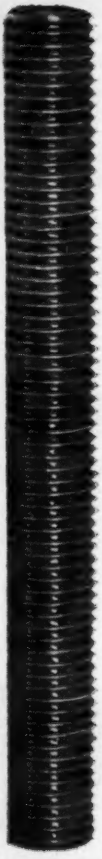

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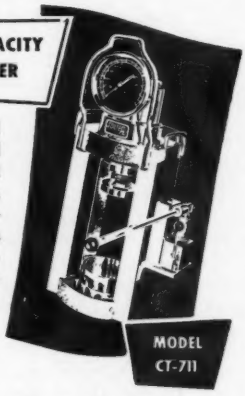

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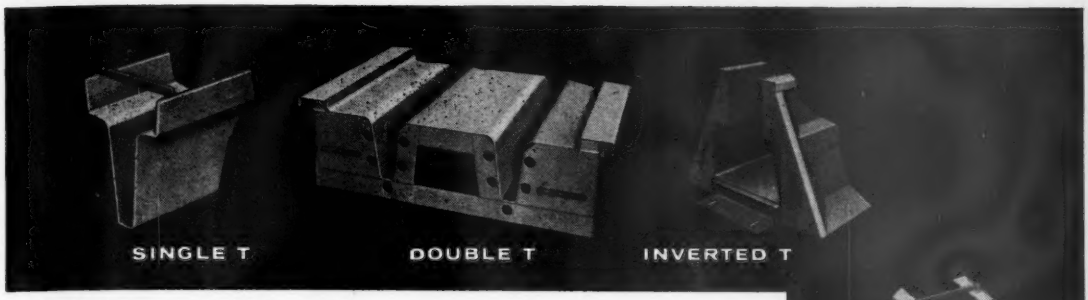

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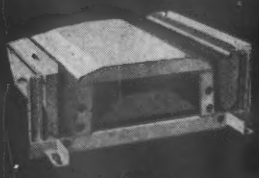
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